APPLICATION OF PALM OIL CLINKER AS FINE AGGREGATE REPLACEMENT IN STONE MATRIX ASPHALT MIX DESIGN

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FACULTY OF ENGINEERING
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APPLICATION OF PALM OIL CLINKER AS FINE AGGREGATE REPLACEMENT IN STONE MATRIX ASPHALT MIX DESIGN

ABSTRACT

The palm oil industry is growing annually which results in a large number of waste materials that are disposed in landfills. To decrease the adverse effects of the wastes on the environment and to improve several properties of asphalt pavement mixture, conventional aggregate materials in the production of asphalt pavement was replaced with palm oil clinker (POC). This leads to green, sustainable, and preserved nature where the dependency on the aggregates from natural sources can be reduced. This research aimed to investigate the effect of POC as a replacement for fine aggregate utilisation in stone matrix asphalt (SMA) design. Six categories of asphalt mixtures were prepared with different POC content at various levels namely, 0%, 20%, 40%, 60%, 80%, and 100%. To determine the Marshall properties and optimum binder content, asphalt mixture samples with different percentage of asphalt binder content (5.0%, 5.5%, 6.0%, 6.5%, and 7.0%) were prepared. Subsequently, optimum bitumen content was used to prepare other sets of asphalt mixture samples to evaluate the mechanical characteristics of the asphalt mixtures. The mechanical properties of SMA mixtures with different percentages of POC replacement were investigated by analysing the resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability, and indirect tensile fatigue. The results showed that the POC is suitable as a substitute of fine aggregates in SMA mixture up to 100% and fulfil the mixture specification criteria concerning the Marshall stability, flow, quotient and volumetric properties. However, the percentage of POC substitution of fine aggregates was has merely influenced the optimum binder content. Based on the overall SMA mixture mechanical properties, the POC can be substituted as fine aggregates up to 100%. The suggested POC optimum was 60% where the SMA

mixtures showed the best performance. In summary, the use of POC in pavement construction is feasible and can decrease the pollution issue related to mass disposal of POC.

Keywords: Asphalt pavement, stone matrix asphalt, Marshall properties, mechanical properties, waste materials, palm oil clinker, sustainable pavement

PENGGUNAAN KLINKER MINYAK SAWIT SEBAGAI PENGGANTI AGGREGAT HALUS DALAM REKABENTUK CAMPURAN ASFALT MASTIK BATU

ABSTRAK

Industri minyak sawit diramalkan akan meningkat setiap tahun yang mengakibatkan sejumlah besar bahan buangan dibuang di tapak pelupusan sampah setiap tahun. Untuk mengurangkan kesan buruk bahan buangan ini ke atas alam sekitar dan untuk meningkatkan beberapa sifat campuran turapan asfalt, ideanya adalah dengan menggantikan bahan aggregat konvensional dalam pengeluaran turapan asfalt. Ini mendorong ke arah pengekalan alam semula jadi, kehijauan dan kelestarian dengan mengurangkan pergantungan kepada aggregat dari sumber asli. Kajian ini bertujuan untuk mengkaji kesan menggunakan batu hangus kelapa sawit (POC) sebagai pengganti agregat halus dalam reka bentuk asfalt batu mastik (SMA). Enam kumpulan campuran asfalt telah disediakan dengan menggunakan peratusan penggantian kandungan POC yang berbeza iaitu 0%, 20%, 40%, 60%, 80%, dan 100%. Untuk menentukan sifat Marshall dan memilih kandungan pengikat yang optimum, sampel-sampel campuran asfalt dengan peratusan yang berlainan (5.0%, 5.5%, 6.0%, 6.5%, dan 7.0%) telah disediakan. Kemudian, dengan menggunakan kandungan bitumen optimum, set sampel campuran asfalt disediakan untuk menilai sifat-sifat mekanikal campuran. Ciri-ciri mekanikal campuran SMA dengan penggantian POC yang berbeza diselidik melalui modulus berdaya tahan, pengesanan roda, rayapan dinamik, kekuatan tegangan tidak langsung, ketahanan Cantabro, dan ujian kelesuan tegangan tak langsung. Keputusan menunjukkan bahawa POC adalah sesuai digunakan sebagai pengganti aggregat halus sehingga 100% dalam campuran SMA dan memenuhi keperluan reka bentuk campuran dari segi sifat-sifat kestabilan Marshall, aliran, kuantiti dan volumetrik. Walau

bagaimanapun, peratusan penggantian POC sebagai aggregat halus mempengaruhi kandungan pengikat yang optimum. Berdasarkan prestasi sifat mekanik keseluruhan, ia menunjukkan bahawa POC adalah sesuai digunakan sebagai pengganti agregat halus sehingga 100%. POC optimum sebagai penggantian agregat halus adalah dicadangkan 60%. Ringkasnya, penggunaan POC dalam pembinaan turapan menunjukkan bahawa ia adalah salah satu teknik dan kaedah yang boleh dilaksanakan untuk mengurangkan masalah pencemaran yang berkaitan dengan pelupusan POC.

Kata kunci: turapan asfalt, asfalt batu mastic, sifat-sifat Marshall, sifat-sifat mekanikal, bahan buangan, batu hangus kepala sawit, turapan lestari.

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LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO	:	American Association of State Highway and Transportation Officials
Al_2O_3	:	Aluminium oxide
%	:	Percent
°C		Degrees Celsius
ANOVA	:	Analysis of Variance
AS	:	Australian Standard
ASTM	:	American Society for Testing and Materials
BS	:	British Standard
CDM	:	Compact density meter
CL	:	Cantabro loss
CaO	:	Calcium oxide
Chl	:	Chloride
cm	:	Centimetre
cps	:	Centipoises
CR	:	Crumb rubber
DSR	:	Dynamic Shear Rheometer
et al.	:	And others
EVA	:	Ethyl-vinyl-acetate
FA	:	Fly ash
FHWA	:	Federal Highway Administration
Fe_2O_3	:	Iron oxide
g	:	Grams
G*	:	Complex shear modulus
GDP	:	Gross Domestic Product
h	:	Hour
HMA	:	Hot Mix Asphalt
JKR	:	Jabatan Kerja Raya (Public Works Department)
K_2O	:	Potassium oxide

KN	:	Kilonewton
KPa	:	kilopascal
MgO	:	Magnesium oxide
mm	:	Millimetre
MnO	:	Manganese oxide
Mpa	:	Megapascal
MPOB	:	Malaysian Palm Oil Board
N	:	Newton
Na_2O	:	Sodium oxide
NCAT	:	National Center for Asphalt Technology
OBC	:	Optimum Binder Content
OGFC	:	Open-graded friction course
Pa	:	Pascal
PAV	:	Pressure aging vessel
PE	:	Polyethene
PG	:	Performance Grade
POC	:	Palm Oil Clinker
POC-0	:	Asphalt mixture containing 0% fine palm oil clinker
POC-20	:	Asphalt mixture containing 20% fine palm oil clinker
POC-40	:	Asphalt mixture containing 40% fine palm oil clinker
POC-60	:	Asphalt mixture containing 60% fine palm oil clinker
POC-80	:	Asphalt mixture containing 80% fine palm oil clinker
POC-100	:	Asphalt mixture containing 100% fine palm oil clinker
POFA	:	Palm Oil Fuel Ash
PSI	:	Pounds per square inch
PVC	:	Polyvinyl chloride
P_2O_5	:	Phosphorus pentoxide
R	:	Pearson correlation coefficient
R2	:	Coefficient of determination
RTFO	:	Rolling Thin Film Oven
SBS	:	Styrene-butadiene-styrene

SGM	:	Specific gravity meter
SHRP	:	Strategic Highway Research Program
SiO_2	:	Silicon dioxide
SMA	:	Stone Matrix Asphalt
SO_3	:	Sulfur trioxide
SPSS	:	Statistical package for the social sciences
St	:	Tensile strength
SUPERPAVE	:	Superior Performing Asphalt Pavement
TiO_2	:	Titanium dioxide
TSR	:	Tensile Strength Ratio
UM	:	University of Malaya
UMATTA	:	Universal Materials Testing Apparatus
VFA	:	Void Filled with Asphalt
VIM	:	Voids in mix
VMA	:	Voids in Mineral Aggregate
XRF	:	X-ray fluorescence
δ	:	Phase angle
µm	:	Micrometre
%	:	Percent
°C	:	Degrees Celsius

CHAPTER 1: INTRODUCTION

1.1 Background

Asphalt concrete is one of the best materials used to pave durable and smoother roads. In general, the asphalt mixture consists of three main materials which are aggregate, asphalt binder and filler. Approximately 94-96% of the mixture total weight is contributed by aggregate and filler, while the remaining 4-6% is asphalt binder. The aggregate influences the pavement quantity more than the other components, which lead to increased demand for aggregate usage in the road construction which requires sourcing of aggregates from other materials.

Recently, depletion of natural resources at a faster rate was noted as people are consuming a large amount of raw materials daily. With the growth of the human population, increased generation of waste was reported due to the increasing number of agricultural and manufacturing industries. This indirectly resulted in a greater amount of waste materials disposed in the landfills every year which has gained research interests to decrease the negative effects of waste on the environment. One of the strategies includes the use of waste materials to replace the original materials such as aggregates in the development of road pavement. This effort may contribute to a green, sustainable and environment-friendly road pavement development by reducing the destruction of natural resources with proper management of solid waste. Previous studies have used waste materials for the design of asphalt pavement by replacing the original aggregate which include waste glasses (Su & Chen, 2002; Jasim, 2014; Shafabakhsh & Sajed, 2014; Androjić & Dimter, 2016; Arabani & Pedram, 2016; Zakaria et al., 2017), steel slag (Wu, Xue, Ye, & Chen, 2007; Pasetto & Baldo, 2011; Ameri & Behnood, 2012; Behnood & Ameri, 2012; Haritonovs, Zaumanis, Brencis, & Smirnovs, 2012; Chen et al., 2015; Chen, Wu, Xiao, Zhao, & Xie, 2016), palm oil kernel shells (Ndoke, 2006; Oyedepo, Olanitori,

& Olukanni, 2015), waste plastics (Rahman & Wahab, 2013; Modarres & Hamedi, 2014b), concrete construction demolition waste (Pourtahmasb & Karim, 2014a; Gómez-Meijide, Pérez, Airey, & Thom, 2015; Fatemi & Imaninasab, 2016; Ossa, García, & Botero, 2016; Nwakaire et al., 2020), recycled asphalt pavement and shingle (Falchetto, Montepara, Tebaldi, & Marasteanu, 2012; Nam, Maherinia, & Behzadan, 2014; Abreu, Oliveira, Silva, & Fonseca, 2015; Colbert, Hasan, & You, 2016; Tapsoba, Baaj, Sauzéat, Di Benedetto, & Ech, 2016), waste tire rubber (Selim, Muniandy, & Abdelrahman, 2005; Ganjian, Khorami, & Maghsoudi, 2009; Mashaan, 2016a), coal bottom ash (Colonna, Berloco, Ranieri, & Shuler, 2012; Pandey & Tare, 2016; Yoo, Park, & Vo, 2016), marble waste (Akbulut & Gürer, 2007), basalt waste (Karakuş, 2011) and mining waste (Gautam, Kalla, Nagar, Agrawal, & Jethoo, 2018). The use of waste materials in asphalt pavement minimises construction cost and preserves nature by reducing the need to harvest natural aggregates from sources (Bolden, Abu-Lebdeh, & Fini, 2013).

Malaysia, Indonesia, and Thailand are the main palm oil producers in the world contributing to almost 91% of global palm oil demand as reported in 2015. Specifically, Malaysia, being one of the largest producers and manufacturers of palm oil products contributes to 32% of the world supply of palm oil (Lam, Jamalluddin, & Lee, 2019). Malaysia alone has produced 19.86 million tonnes of palm oil and exported 18.47 million tonnes of palm oil in 2019 (Malaysian Palm Oil Board, 2019). Malaysia's palm tree planting is constantly increasing in line with the government's strategy to produce palm oil biodiesel. The Malaysian Palm Oil Board (MPOB) declared that the area of land designated for oil palm cultivation in 2018 was about 5.8 million hectares (Malaysian Palm Oil Board, 2018).The palm oil industry greatly influences the economy of Malaysia as a major contributor to the gross domestic product (GDP). In 2018, the palm oil industry contributes about RM80 billion representing 8.7% of total GDP (Andiappan, Ng, & Tan, 2019).

Palm oil production generates various types of waste materials and by-products such as empty fruit bunches, palm oil mill effluent, steriliser condensate, fibres and kernel shells as shown in Figure 1.1 (Loh, 2017). Palm oil processing activities were reported to produce only 10% of oil from fresh fruit and kernel while the remaining 90% is the underutilised biomass or waste (Dungani et al., 2018).

Figure 1.1: Biomass by-products generation in the palm oil industry

Millions of tonnes of annual wastes produced by the palm oil industries contribute to several pollution issues. The amount of waste is expected to increase yearly due to the continuous global consumption of palm oil (Mohammed et al., 2011). Figure 1.2 illustrates the estimated annual summary of the palm oil industry biomass generation in Malaysia in 2017. It was reported that Malaysia produces about 50 million tonnes of palm oil biomass every year and it is expected to be 100 million tonnes in 2020 (Aghamohammadi et al., 2016)

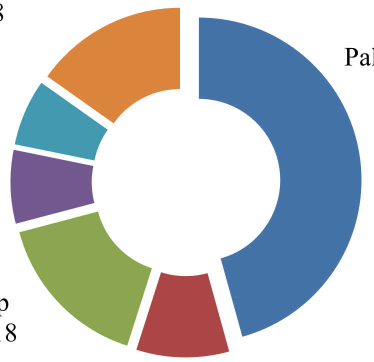


Figure 1.2: Palm oil biomass produced in Malaysia (millions of tonnes) (Hamzah, Tokimatsu, & Yoshikawa, 2019)

The oil palm biomass such as shells and fibres were used as boiler fuel for power generation purpose (Hamzah et al., 2019; Yusof et al., 2019). This process in turn creates a large amount of new waste materials which are palm oil clinker (POC) and palm oil fuel ash (POFA) as shown in Figure 1.3.

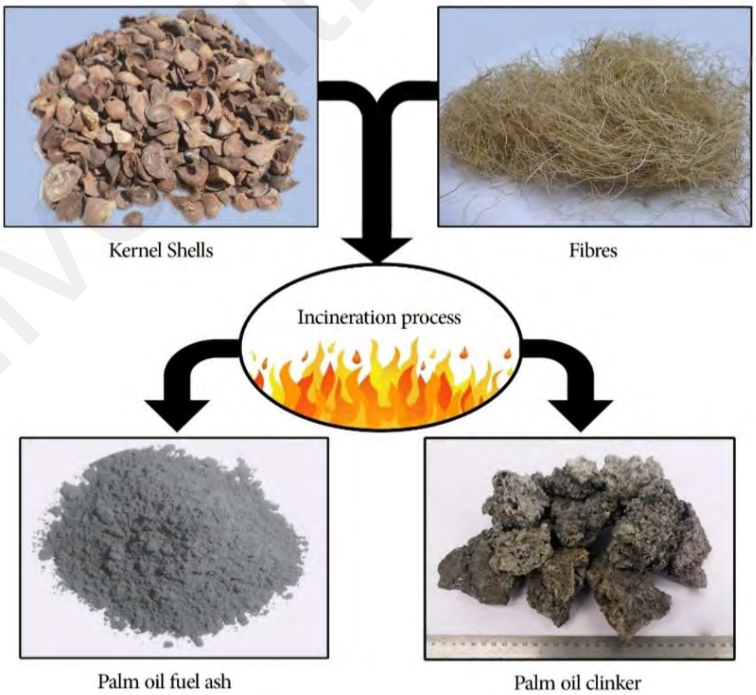


Figure 1.3: By-products obtained after incineration of oil palm waste materials

In Malaysia, a high amount of POC and POFA are generated in palm oil mills as waste material with insignificant return. So, those wastes can be converted into potential construction materials such as an aggregate or cement replacer in concrete construction with engineering potentials and economic advantages. Based on literature search findings, POC has been studied by many researchers as an aggregate replacer concrete construction which consider as lightweight material that can reduce a dead load of concrete structures and keep the strength of concrete at a suitable level (Kanadasan et al., 2015; Ibrahim & Razak, 2016; Ibrahim, Razak, & Abutaha, 2017; Nayaka, Alengaram, Jumaat, Yusoff, & Ganasan, 2019). Moreover, POFA was used in concrete construction to improve its properties such as resisting chloride attack (Chindaprasirt, Rukzon, & Sirivivatnanon, 2008), increasing the drying shrinkage (Tangchirapat & Jaturapitakkul, 2010), resisting sulphate attack (Jaturapitakkul, Kiattikomol, Tangchirapat, & Saeting, 2007), and acting as a cement replacer (Alsubari, Shafigh, Jumaat, & Alengaram, 2014; Khankhaje et al., 2018; Arif, Asrah, Rizalman, & Dullah, 2019; Hosen, Jumaat, Alengaram, Sulong, & Alsubari, 2019). However, those waste materials can be transformed into a potential alternative aggregates or fillers that can be useful in road pavement construction.

1.2 Problem Statement

Recently, the usage of natural aggregates increased dramatically due to rapid growth and civilisation worldwide. Natural resources deplete faster due to the greater amount of raw materials consumption. The depletion of primary aggregates leads to increase in the awareness to protect the environment and preserve nature by reducing the harvest of natural aggregates (Bolden et al., 2013). Besides, aggregate natural resource scarcity is also an issue that can affect future generations.

Human population growth indirectly causes an increase in agricultural and manufacturing industries to meet the consumer demand for various products which in turn led to a greater amount of waste materials disposed in the landfills every year. However, the disposal of solid agricultural and industrial wastes in developed countries is of serious concern

(Hamada et al., 2020). The palm oil industry contributes to the pollution problem where millions of tonnes of wastes are produced annually (Mohammed et al., 2011; Aghamohammadi et al., 2016) which has been projected to increase with the ongoing global consumption demand for palm oil.

Continuous research efforts have been dedicated to reduce the negative environmental effects of wastes which includes the use of waste materials to replace conventional materials. This leads to a reduction in the harvest of natural aggregates, environmental pollution from residential and agricultural waste materials besides saving the energy in overall.

To date, limited studies are available on the use of oil palm waste in asphalt mixtures. Based on the literature review search, no reports were found on the utilisation of palm oil clinker (POC) as an aggregate substitute in the asphalt pavement mixture. Therefore, this study aimed to investigate the potential application of POC as the aggregate replacer in the asphalt mix design. The findings from this study are expected to provide an insight into the usage of POC as a substitute to reduce the use of natural aggregates, reduce environmental pollution due to waste material disposal, minimise the construction cost, and increase the mechanical performance of asphalt mixture.

1.3 Research Gap

In Malaysia, a high amount of POC is generated in palm oil mills as waste material with an insignificant return. However, POC can be transformed into a potential alternative material that can be useful in the construction industry. Based on literature search findings, POC has been studied by many researchers as an aggregate replacer in concrete construction. Nevertheless, the use of POC as an aggregate in asphalt pavement mixture has not been reported. Therefore, this study aimed to assess the use of POC waste as an aggregate in SMA. Mechanical performance of SMA mixture containing different percentages of POC as a fine aggregate replacer was investigated. The efficient utilisation of POC could impact the environment positively. From the perspective of environmental friendliness and sustainability, the utilisation of POC from oil palm wastes is a promising project.

1.4 Research Aim and Objectives

This research aimed to study the effect of POC as a fine aggregate replacer on the mechanical properties of Stone Matrix Asphalt (SMA) pavement design. Following are the four objectives to achieve the research aim:

1. To identify the effect of POC as a fine aggregate replacer on stability, flow and volumetric properties of SMA mixtures.
2. To determine the optimum binder content of SMA mixtures with the incorporation of different proportion of POC.
3. To evaluate the effect of POC as a fine aggregate replacer on the mechanical properties performance of SMA mixtures.
4. To propose the optimum POC proportion as a fine aggregate replacer for SMA mix design.

1.5 Novelty and Contributions of the Study

The present study contributes to two key domains which are: (i) body of knowledge, and (ii) industry or practice which are summarised and presented in Figure 1.4.

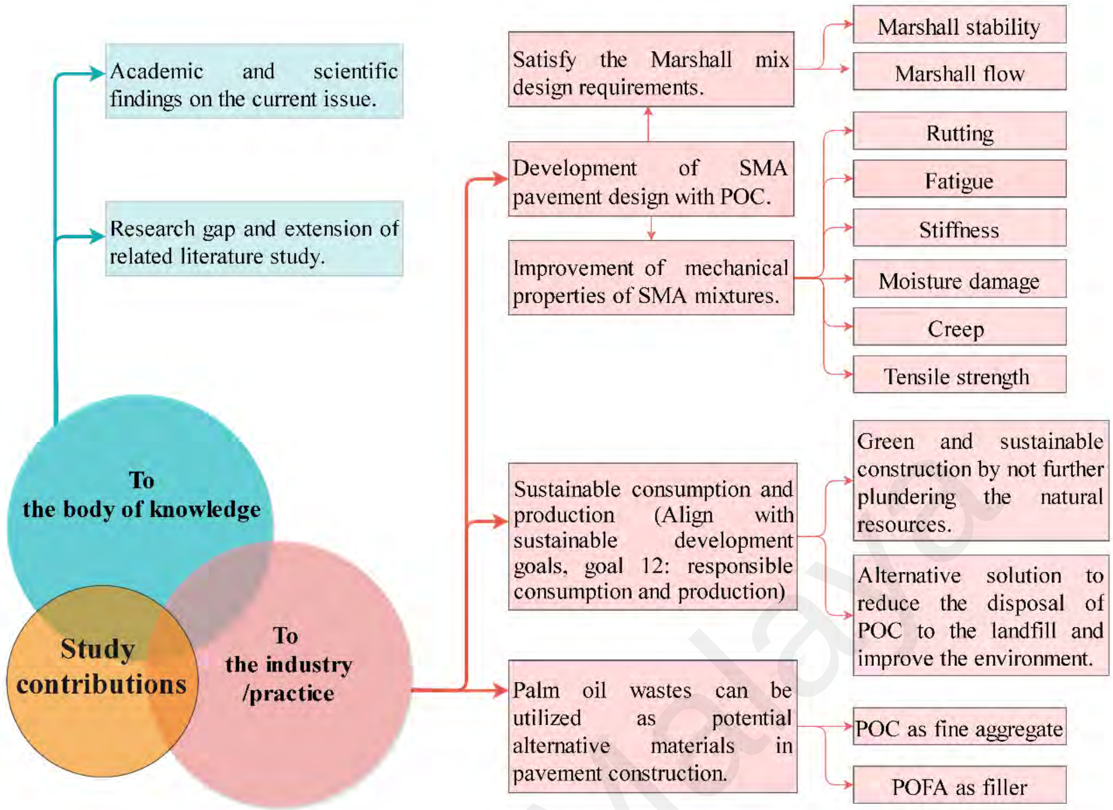


Figure 1.4: Thesis contribution diagram

For the body of knowledge, the findings are expected to enhance the understanding and provide academic and scientific information about the use of oil palm wastes such as POC and POFA in SMA mixture besides raising awareness on SMA application in Malaysian construction pavement. Moreover, the adoption and the use of POC as a fine aggregate replacer in asphalt mix design will be useful for other researchers as there are very limited related studies that have been investigated.

For industry or practice domain, this study contributes in the development of SMA pavement design with oil palm wastes using Marshall mix design besides the identification of optimal asphalt binder content for each group of POC aggregate replacer. The findings could suggest the use of POC in improving the mechanical properties of SMA mixture such as rutting, fatigue, stiffness, moisture damage, creep, and tensile strength. The improved properties may extend the asphalt pavement lifecycle and reduce the maintenance cost. Moreover, the use of POC in pavement construction is one of the technical and feasible methods to decrease the pollution issue related to the disposal of POC. Also, further plundering of natural resources can be overcome where the proposed SMA pavement design is in line with the Sustainable Development Goal 12: responsible consumption and production. Another relevant contribution of this study is that oil palm wastes (POC and POFA) can be utilised as potential alternative aggregates or fillers for road pavement development.

1.6 Study Scope

This research investigates the mechanical performance of SMA mixture with different POC content as a fine aggregate replacer. The aggregate gradation of SMA20 was designed according to specifications by the Malaysian Public Works Department for road works. All the materials used in this study were purchased from local suppliers. Aggregates and quarry dust were procured from Kajang Rock Sdn. Bhd. while asphalt binder 80/100 were obtained from Asphalt Technology Sdn. Bhd. The materials were stored in highway laboratory. The physical properties of 80/100 asphalt binder and granite aggregates were evaluated to confirm their qualities. Six different proportions of POC (0%, 20%, 40%, 60%, 80% and 100%) by the total mix weight of fine aggregates were used as a fine aggregate substitute in SMA mixtures. Additionally, 50% of POFA was used as filler in the POC asphalt mixtures. To find the optimum binder content for different mixtures, the Marshall and volumetric properties analysis was performed. Moreover, to evaluate the mechanical properties performance, SMA20 mixture with different proportion of POC replacement was investigated by analysing the resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability, and indirect tensile fatigue. All the performance tests were carried out on Marshall samples with 102 mm of diameter and 65 mm thickness except for the wheel tests, the analysis was carried out on slab samples with a dimension of 200 mm × 300 mm × 50 mm. All the tests were carried out according to American Society for Testing and

Materials (ASTM), American Association of State Highway and Transportation Officials (AASHTO), Australian Standard and British Standard and details of each test are as explained in Chapter 3. All the laboratory analyses were performed in the Highway Laboratory, Civil Engineering Department, Engineering Faculty, University of Malaya, Malaysia.

1.7 Organisation of the Thesis

The thesis consisted of five chapters as the following:

Chapter One: This chapter presents a brief introduction, problem statement, research gap, objectives and scope of the study.

Chapter Two: This chapter summarises various literature related to the research topic. For instance, hot mix asphalt method with a detailed discussion of its components, waste materials used for pavement construction, and application of various waste materials as a modifier, aggregate, and filler were reviewed. Information on oil palm industry and the associated wastes are also discussed.

Chapter Three: This chapter focuses on the experiments and testing methods that were used in this research to determine the physical properties of materials. Besides, basic and detailed approaches of mix design and method performance tests on SMA samples that were carried out to evaluate the properties asphalt mixtures are also explained.

Chapter Four: This chapter presents and discusses in detail experimental results of (i) materials physical properties, (ii) Marshall mix design, (iii) optimum asphalt binder for different SMA mixtures, and (iv) mechanical performance of SMA mixtures.

Chapter Five: This chapter summarises the study and significant findings. Finally, references and related appendices are included at the end.

CHAPTER 2: LITERATURE REVIEW

This literature review chapter covers waste materials that were used in asphalt pavement construction and their effect on the performance of asphalt mixtures. Waste materials used in pavement construction may emerge from a wide variety of sources. Waste materials can be classified based on their use in asphalt pavement, namely, asphalt modification, filler, and aggregate as described in the following sub-sections.

2.1 Waste Materials Used as Asphalt Modifier

Neat or virgin asphalt binder is not suitable for some regions where high-temperature condition prevails, besides high and slow traffic roads and stationary traffic at intersections. Therefore, the asphalt binder needs to be modified to meet the requirements for different weather conditions. One of the methods used is to include waste material to modify the binder. Many studies indicated that the use of waste materials in asphalt binder as a modifier improved the asphalt binder rheological properties. Softening point and penetration tests showed high resistance to fatigue cracking and rutting (Fernandes, Costa, Silva, & Oliveira, 2015; Appiah, Berko-Boateng, & Tagbor, 2017). Different waste materials used as asphalt binder modifier are discussed in the following sub-sections.

2.1.1 Crumb Rubber

Crumb rubber (CR) is one of the waste materials that is available to modify asphalt binder. Malaysian scrap tires production approaches 10 million tiers per year and is generally disposed in an environmentally unfriendly manner (Ibrahim, Katman, Karim, & Mahrez, 2009). CR has been used as one of the common additives to improve the rheological properties of asphalt binder (Kök & Çolak, 2011; Ghasemi & Marandi, 2013; Mashaan, Ali, Koting, & Karim, 2013). The capability of CR in enhancing the asphalt binder properties depends on several variables. Internal variables include quantity, type, and particle size of crumb rubber, besides the source and type of asphalt binder. Meanwhile, external variables that can affect the performance of CR include blending process parameters such as the mixing time, shear rate, temperature, and mixing process type (dry or wet) (Lee, Akisetty, & Amirkhanian, 2008; Magar, 2014). For the wet process, CR is used to modify asphalt binder which was then added into the asphalt mixture. The particles of CR swell in asphalt binder mixture by absorbing a part of lighter fractions of binder followed by the formation of a viscous gel leading to physical changes. Researchers have demonstrated that addition of CR into pure asphalt binder can produce modified asphalt mixture with improved resistance to thermal cracking, fatigue cracking, and rutting besides reducing the asphalt mixture thickness (Raad & Saboundjian, 1998; Huang, Mohammad, Graves, & Abadie, 2002; Fontes, Trichês, Pais, & Pereira, 2010). In contrast, for the dry process, the aggregate is mixed with CR before the addition of asphalt binder into the asphalt mixture.

Recently, Wulandari and Tjandra (2017) examined the use of CR as an additive in asphalt binder using the wet process. Two different CR concentrations which were 1% and 2% and two different CR sizes which were #40 and #80 were investigated. Interestingly, the incorporation of CR into asphalt binder improved asphalt mixture strength and consistency.

In another study, Bansal, Misra, and Bajpai (2017) modified the asphalt binder with combinations of different concentrations of CR (5%, 10% and 15%) and waste plastic (4%, 6%, 8% and 10%). Laboratory testing results showed that the strength of asphalt mixture increased by 50% as compared to the control mix. Furthermore, the environmental issue caused by the disposal of these waste materials can be reduced indirectly.

Mashaan, Ali, Karim, and Abdelaziz (2011) investigated the effect of different CR concentrations on physical and rheological properties of asphalt binder. The modified asphalt binders were prepared using five different concentrations of CR (4%, 8%, 12%, 16% and 20%). The results showed that the inclusion of CR increased the elastic recovery and decreased the ductility and penetration of asphalt binder. Moreover, the rheological property of asphalt binder was found to improve with higher CR content which suggests its resistance to rutting deformation which may occur in the asphalt layer as a result of increased traffic loading.

In a different study, Kebria, Moafimadani, and Goli (2015) investigated the effect of CR addition on the rheological behaviour of asphalt binder. The modified binders were prepared in the laboratory by blending five different concentrations of CR ranging from 4% to 20% with an interval of 4% with 60/70 penetration grade of asphalt binder. The modified binders were tested using conventional and Strategic Highway Research Program (SHRP) tests. The results showed that the binder which was modified with greater than 8% CR resulted in one- or two-degree enhancement in the asphalt binder performance. Besides, there was an increment in the ($G^*/\sin\delta$) value at several temperatures which indicated that the modified binder was more resistant to rutting.

Particles size of CR is one of the important factors that control the properties of CR-modified asphalt. Based on the sizes, the CR can be divided into four categories which are coarse (9.5 mm - 6.3 mm); medium (2 mm - 0.6 mm); fine (0.4 mm - 0.18 mm); and superfine (0.15 mm - 0.075 mm) (Leite & Soares, 1999; Sunthonpagasit & Duffey, 2004). In a later study, Chiu and Lu (2007) investigated the feasibility of using asphalt rubber produced by blending CR of two different particle sizes which were produced in Taiwan with an asphalt binder. It was found that it was not feasible to produce a suitable SMA mixture using an asphalt rubber by blending an AC-20 with 30% of coarse CR with a maximum size of 0.85 mm. However, typical volumetric requirements for SMA were met when an asphalt rubber containing 20% of fine CR with a maximum size of 0.6 mm was used. In another study, Magar (2014) evaluated the performance of asphalt binder modified with different sizes of 15% of CR. Four different categories of CR were used, which were coarse (1 - 0.6 mm); medium (0.60 - 0.30 mm); fine (0.30 - 0.150 mm); and superfine (0.150 - 0.075 mm). The results showed that the best size to be used for CR modification is 0.3 - 0.15 mm for commercial production of CR-modified asphalt binder.

2.1.2 Plastics

Plastics are organic polymers of high molecular mass and often contain other substances. Mainly there are two types of polymers which are elastomers and plastomers. An elastomer is a polymer such as styrene-butadiene-styrene (SBS) which possess the elastic or rubbery characteristic. Upon the release of load, the elastomer can recover to its original shape and size and cause the binder to be more flexible. Meanwhile, a plastomer is a polymer such as polyethene (PE), ethyl-vinyl-acetate (EVA), and polyvinyl chloride (PVC). Plastomer polymers can attain a very high strength which can endure heavy loads (Isacsson & Lu, 1995; Liu, 1998). The elastomers are usually used to extend both minimum and maximum service temperatures, whereas plastomers are recognised as effective additives at high service temperatures (Lu, Isacsson, & Ekblad, 1998; Kök & Çolak, 2011; Ameri, Mansourian, & Sheikhmotevali, 2013).

Many researchers reported that utilisation of waste plastic as a modifier in the asphalt binder can increase the softening point values and decrease the penetration values which indicate high resistance to fatigue cracking and rutting (Köfteci, Ahmedzade, & Kultayev, 2014; Fernandes et al., 2015; Appiah et al., 2017). Köfteci et al. (2014) modified asphalt binder by using waste material sources such as windows, blinds, and cable waste plastics. To evaluate the modified asphalt binder properties, the asphalt binder samples were tested through rotational viscometer, dynamic shear rheometer and bending beam rheometer. The results showed that the addition of waste materials originated from windows, blinds and plastics increased the performance of modified asphalt binder by 1 - 3% at high temperatures. For low temperatures, the plastic waste did not affect the performance of asphalt binder whereas cable wastes improved the performance of asphalt binder by 5% at low temperatures.

2.1.3 Oils

As stated in previous studies, the performance of the asphalt binder can be improved by using different types of available waste materials. Waste oils have become one of the environmental pollutants and are seriously affecting lives. Therefore, waste oil can be raw material in a large quantity for asphalt pavement. The flashpoint of waste oil is above 220 °C which indicates waste oil has a high safety value and can be applied in asphalt mixtures. Many researchers have shown that waste oils can be used to rejuvenate aged asphalt binders and produce bio-asphalt. There are many types of waste oils which can be used to modify asphalt binders or to produce bio-asphalt such as cooking oil (Asli, Ahmadinia, Zargar, & Karim, 2012; Zargar, Ahmadinia, Asli, & Karim, 2012; Wen, Bhusal, & Wen, 2013; Su, Qiu, Schlangen, & Wang, 2015; Azahar, Jaya, Hainin, Bujang, & Ngadi, 2016), frying oil (Eriskin, Karahancer, Terzi, & Saltan, 2017), vegetable oil (Chen, Leng, Wu, & Sang, 2014; Al-Omari, Khedaywi, & Khasawneh, 2018), castor oil (Zeng, Pan, Zhao, & Tian, 2016), maize oil (Portugal, Lucena, Lucena, Costa, & de Lima, 2017) and palm kernel oil (Alamawi, Khairuddin, Yusoff, Badri, & Ceylan, 2019).

Wen et al. (2013) used waste cooking oil-based bioasphalt as an alternative binder for asphalt pavement. Both binder and mixture tests were carried out to evaluate the performance of waste cooking oil-based bio-asphalt as a binder. Waste cooking oil was blended with an original binder at three different concentration levels (10%, 30% and

60%). The findings of binder analysis showed that the use of waste cooking oil-based bio-asphalt decreased the resistance to fatigue cracking and rutting. Meanwhile, mixture performance analysis indicated that the stiffness of mix reduced by the addition of bioasphalt. Furthermore, moisture susceptibility test findings showed that bioasphalt mixes passed the minimum tensile strength ratio requirement.

In a more recent study, Eriskin et al. (2017) used waste frying oil as a modifier to decrease the amount of asphalt binder required in the asphalt pavement which can indirectly reduce construction cost. Waste frying oil was mixed with the asphalt binder at three different percentages (1%, 3%, and 5%). The results showed that the optimum asphalt binder content of asphalt mixture decreased from 5.13% to 4.58%. Besides, the penetration value increased by 2.4 times while the softening point decreased by 0.82 times as a result of asphalt binder modification with waste frying oil.

2.1.4 Nano-materials

Nano-materials that have a large surface area and very small size (1 to 100 nm) can be used as asphalt modifier and can improve both rheological properties of asphalt binder and engineering properties of asphalt mixture (Jeffry et al., 2018). Many types of nano-materials were used to modify asphalt binders such as coconut shell ash (Jeffry et al., 2018), rice husk ash (Ramadhansyah, Irwan, et al., 2019), palm oil ash (Rusbintardjo, Hainin, & Yusoff, 2013), silica (Saltan, Terzi, & Karahancer, 2017), carbon fibre (Khattak, Khattab, Rizvi, & Zhang, 2012), and rattan and clay (You et al., 2011).

2.2 Waste Materials Used as a Filler

Mineral filler is an aggregate of the smallest particle which can pass through a 0.075 mm sieve and the mineral filler is being used in asphalt mixtures. The filler usually consists of finely divided materials such as stone dust, Portland cement, slate dust, and hydrated lime. It has two important roles in asphalt mixture: (i) to fill the voids between individual particles of aggregates to obtain denser and stiffer layers, and (ii) to provide more contact points between the aggregates. However, the fine particles of mineral filler may sometimes assume a dual role. Extremely fine particles are located in the asphalt films which can coat coarser aggregate particles. In this case, mineral fillers can affect the performance of asphalt mixtures including permanent deformation, fatigue cracking, and moisture susceptibility (Kandhal, Lynn, & Parker, 1998). To date, various waste materials and natural stones were used as mineral filler in asphalt pavement such as palm oil fuel ash (Kamaluddin, 2008; Borhan, Ismail, & Rahmat, 2010; Ahmad et al., 2012; Maleka, Hamad, & PutraJaya, 2014), fly ash (Ali, Chan, Simms, Bushman, & Bergan, 1996; Tapkın, 2008; Mistry & Roy, 2016), brake pad (Hu, Wang, Pan, & Bai, 2017), rice husk ash (Sargın, Saltan, Morova, Serin, & Terzi, 2013; Jaya et al., 2018; Ramadhansyah, Haryati, et al., 2019; Ramadhansyah, Irwan, et al., 2019), marble waste dust (Karaşahin & Terzi, 2007), glass powder (Jony, Al-Rubaie, & Jahad, 2011; Ghasemi & Marandi, 2013; Saltan, Öksüz, & Uz, 2015), andesite waste (Uzun & Terzi, 2012), asphaltite (Yilmaz, Kök, & Kuloğlu, 2011), basalt (Karakuş, 2011), waste ceramic materials (Huang, Dong, & Burdette, 2009), hydrated lime (Aragão, Lee, Kim, & Karki, 2010), coal waste powder (Modarres & Rahmanzadeh, 2014), cleaned oil-drill cuttings (Dhir, Csetenyi, Dyer, & Smith, 2010), red mud (Zhang et al., 2018; Zhang et al., 2019), sawdust (Al-Khateeb, Khedaywi, & Irfaeya, 2018) and oil shale ash (Wang, Cheng, Tan, & Shi, 2020).

2.2.1 Palm Oil Fuel Ash

Palm Oil Fuel Ash (POFA) is a by-product generated as a result of palm oil processing. After palm oil is extracted from oil palm fruit bunches, oil palm husks and shells that were produced as waste materials were combusted and used as fuel in the palm oil boiler to produce electricity. The typical ratio of oil palm shells to husks used is 20:80 to produce POFA (Borhan et al., 2010). Generally, after the combustion of oil palm waste materials, about 5% of POFA by weight of solid wastes will be produced (Sata, Jaturapitakkul, & Kiattikomol, 2004). The ash produced varies from grey to black based on the carbon content as shown in Figure 2.1.

Figure 2.1: Palm oil fuel ash (Islam, Alengaram, Jumaat, Bashar, & Kabir, 2015).

POFA has been reported to enhance the properties of asphalt mixture (Kamaluddin, 2008; Borhan et al., 2010; Ahmad et al., 2012; Maleka et al., 2014). For instance, Ahmad et al. (2012) suggested the practical use of POFA as a filler at different percentages in asphalt pavement. They found that the addition of 3% of POFA by total weight of aggregate could increase the stability and resilient modulus of asphalt pavement. As reported in an earlier study by Kamaluddin (2008), 50% of POFA by weight of filler content is the optimum value for utilisation of POFA in asphalt pavement mixture as filler material.

2.2.2 Fly Ash

In 2010, pulverised coal was used to generate about 40% of electricity in Malaysia which consumes about 11 million tonnes of coal every year. Coal-burning in thermal power plants was reported to result in the production of fly ash (FA) of about 3 million tonnes per year (Yap, Alengaram, Jumaat, & Foong, 2013). In future, it is predicted that FA production will increase by two times and there will be disposal issue of this waste as it causes environmental and storage problems (Yap et al., 2013). Many studies have shown that FA can be used effectively in asphalt pavement as a filler replacer (Ali et al., 1996; Tapkın, 2008; Mistry & Roy, 2016).

In a more recent study, Mistry and Roy (2016) investigated the effect of dust filler replacement with FA (ranging from 2 to 8%) in asphalt mixture. Different asphalt binder content (3.5 - 6.5% at 0.5% increment level) was used to prepare samples using 2% of hydrated lime. The results showed that the stability value of mixture with lower binder content was higher with 4% of FA found as the optimum filler content in comparison to the control mixture.

2.2.3 Glass Powder

Incorporation of glass in asphalt pavement which is commonly known as glasphalt was first introduced into several international markets in the late 1960s (Shafabakhsh & Sajed, 2014). In recent years, more studies have successfully incorporated glass powder into asphalt mixtures as a filler (Jony et al., 2011; Ghasemi & Marandi, 2013; Saltan et al., 2015).

In 2011, Jony et al. explored the use of glass powder as a filler in asphalt pavement with the glass powder of particle size that can pass through a 0.075 mm sieve. The research involved the use of nine mixtures and three types of fillers (limestone powder, ordinary

Portland cement and glass powder) of different percentages (4%, 7%, and 10%) by total weight of aggregate. The test results showed that 7% of glass powder was the optimum concentration to obtain the highest stability (Jony et al., 2011).

Later in 2015, Saltan et al. (2015) investigated the usage of waste glass powder dust as a filler material in asphalt mixture as an alternative to stone dust. The Marshall mix design method was used to determine the optimum bitumen content using six different asphalt binder contents (4.0% to 6.5% with an increment level of 0.5%). In the study, different percentages (4%, 5%, 6%, 7%, 8%, and 9%) of cullet, glass bottle waste, and stone dust were used as mineral fillers to prepare asphalt mixture samples. Asphalt mixture samples were evaluated using the Marshall stability test and the findings indicated potential use of cullet and glass bottle waste as a mineral filler alternative to stone dust in asphalt mixtures. Besides, usage of glass waste in hot mix asphalt pavements would be very useful for waste management.

2.3 Waste Materials Used as an Aggregate

Various waste materials were used to replace aggregate in asphalt pavement such as glasses, steel slags, palm kernels, plastics, construction and demolition wastes, crumb rubbers, bottom ashes, marble wastes, mining wastes, recycled asphalt pavement and reclaimed asphalt shingles.

2.3.1 Glasses

Waste glasses are one of the waste products that have been filling the landfills as shown in Figure 2.2.

Figure 2.2: Waste glasses

Waste glasses are harmful to our environment as it is not biodegradable and take several millions of years to decompose. In pavement construction, waste glasses are being used as an aggregate replacer. In recent years, several experiments have been performed to integrate waste glasses in asphalt mixtures without affecting their performance (Su & Chen, 2002; Jasim, 2014; Shafabakhsh & Sajed, 2014; Androjić & Dimter, 2016; Arabani & Pedram, 2016; Zakaria et al., 2017).

Zakaria et al. (2017) used plastic and glass as an aggregate substitute in asphalt mixtures. The researchers used three types of glasses: sheet glass, bottle glass and liquid crystal display glass in the study. A concentration of 5% by total weight of asphalt mixture of waste material was used as an aggregate substitute. The modified asphalt mixtures were tested for Marshall and resilient moduli. The findings showed that the asphalt mixture substituted with 1% of recycled plastic and 4% of recycled glass resulted in almost similar performance with that of the control asphalt mixture. The research outcome proposed that recycled glass is feasible to be applied as an aggregate substitute in asphalt paving mixture.

2.3.2 Steel Slag

Steel slag produced in general is very angular and porous with a rough surface texture. Many studies focused on the use of steel slag as an aggregate in asphalt pavement (Wu et al., 2007; Pasetto & Baldo, 2011; Ameri & Behnood, 2012; Behnood & Ameri, 2012; Haritonovs et al., 2012; Chen et al., 2015; Chen, Wu, Xiao, et al., 2016).

Pasetto and Baldo (2011) conducted an experiment in a laboratory where steel slag was used to develop asphalt pavement. Three steel slag mixes and a limestone aggregate mix were compared. Both Marshall and Superpave design methods which used steel slag were found to be compatible with the conventional asphalt mixes. No excessive permanent deformation was noted for the two mix design methods. Overall performance concerning stiffness, fatigue, and sensitivity to moisture damage was good. Mixes with the highest percentage of steel slag (90%) showed the best performance compared to the other steel slag mix concentration levels (0%, 30% and 60%).

A year later, Ameri and Behnood (2012) reported the effect of steel slag as an aggregate replacer on mechanical properties of asphalt pavement. The outcome showed that steel slag can enhance Marshall stability, tensile strength, resilient modulus besides being resistant to permanent deformation and moisture damage of asphalt mixes.

2.3.3 Oil Palm Kernel Shell

Oil palm generates various types of waste materials such as oil palm fibres and oil palm kernel shells as shown in Figure 2.3. Countries such as Malaysia, Indonesia, and Thailand produce almost 91% of the total world’s palm oil in 2015. Malaysia, being one of the largest producers and manufacturers of palm oil-based products contributes about 32% of the world supply of palm oil (Lam et al., 2019). Consequently, Malaysia is also one of the top listed countries with huge resources of oil palm shell (Ahmad, Ibrahim, & Tahir,

2010) with over 4 million tonnes of solid waste of oil palm shell produced per year (Nagaratnam, Rahman, Mirasa, Mannan, & Lame, 2016).

The use of palm kernel shells as aggregates in asphalt mixtures have been investigated by many researchers. Ndoke (2006) analysed the efficiency of palm kernel shells as coarse aggregates on asphalt mixture using Marshal stability and flow tests. It was reported that palm kernel shells can be used to substitute coarse aggregates up to 30% while for high traffic roads, substitution at a level of 10% was proposed.

Figure 2.3: Oil palm kernel shells

Oyedepo et al. (2015) explored the use of the palm kernel shell as a partial substitute for fine and coarse aggregates when developing asphalt pavement. Palm kernel shell was applied at different concentration levels (20%, 40%, 50%, 60% and 80%) by weight of both fine and coarse aggregates. The findings revealed that palm kernel shells can be used as an alternative material for coarse aggregates for low, medium and heavy traffic roads. Meanwhile, 20% of palm kernel shells can be used as fine aggregates for heavy traffic road and 60% of palm kernel shells was proposed for medium road traffic. Moreover, the usage of the palm kernel shell as a substitute for asphalt pavement can reduce environmental pollution due to the large disposal of palm kernel shells. Besides, palm kernel shells can be a source of building material which can reduce the utilisation of natural gross aggregates.

2.3.4 Plastics

As mentioned earlier, plastics were used as asphalt modifier, which can decrease the penetration values and increase the softening point values of modified asphalt binder resulting in high resistance to fatigue cracking and rutting.

Wet and dry procedures are typically used to incorporate plastics in the asphalt mixture. For the wet phase, the plastic is combined with the binder before adding the binder into the mixture. Meanwhile, for the dry phase, plastic is mixed with the aggregate before the application of asphalt binder (Ahmadinia, Zargar, Karim, Abdelaziz, & Shafigh, 2011). The key reason for the preference of dry method over the wet method is the maintenance of the normal state of the plastic in the mixture with the allowance of minor form and property modifications. Many researchers have used the dry process method to replace the aggregate using different waste plastics for asphalt pavement (Ahmadinia, Zargar, Karim, Abdelaziz, & Ahmadinia, 2012; Moghaddam & Karim, 2012; Rahman & Wahab, 2013; Modarres & Hamedi, 2014b; Zakaria et al., 2017).

In a study by Rahman and Wahab (2013), the effect of waste plastics as a partial fine aggregate replacer in asphalt mixture was reported and the optimum waste plastic replacement was determined by evaluating the permanent deformation and stiffness behaviours. The results showed that 20% replacement of fine aggregate with waste plastics can lead to maximum permanent deformation while the stiffness of waste plastic-based asphalt mixtures tends to decrease compared with control asphalt mixture. Therefore, waste plastic utilisation can improve permanent deformation resistance of asphalt mixture. From environmental and economical aspects, waste plastics are suitable for application in the development of road pavements.

Modarres and Hamedi (2014b) investigated the effect of different levels (2 - 10%) of waste plastic bottles on fatigue properties of asphalt mixes at two different temperatures, 5 and 20 ºC, using the indirect tensile test. Besides, the effect of waste plastic bottles was compared to a conventional polymer additive, Styrene Butadiene Styrene (SBS). The findings showed that both additives improved the fatigue response of asphalt mixes. Moreover, the asphalt mixture modified by SBS showed better fatigue behaviour than asphalt mixture modified using waste plastic bottles.

2.3.5 Construction and Demolition Wastes

In general, building structures are constructed using concrete which is made of aggregates, sand, cement and water. Environmental and economic concerns have motivated authorities to use recycled products for new productions. When a concrete building is destroyed or renewed, recycling is of growing interest to reuse the materials from concrete demolition. This indirectly enables the preservation of natural resources for future generations (Poon, Shui, Lam, Fok, & Kou, 2004).

Waste materials resulting from destroyed concrete buildings is one of the world's largest waste which can be used as aggregate in asphalt pavement mixtures (Pourtahmasb & Karim, 2014a; Gómez-Meijide et al., 2015; Fatemi & Imaninasab, 2016; Ossa et al., 2016). Several studies (Poon et al., 2004; Pourtahmasb & Karim, 2014a; Ossa et al., 2016; Tahmoorian & Samali, 2018; Nwakaire et al., 2020) have reported the application of waste construction aggregates in hot mix asphalt. Pourtahmasb and Karim (2014a) examined the potential utilisation of recycled concrete aggregates in pavement mixtures. Three categories of recycled concrete aggregates at various percentages were mixed with natural aggregates to produce asphalt pavement mixtures. The findings showed that the replacement of natural aggregate with waste concrete aggregate required higher asphalt binder content. Based on project demands and traffic level, usage of unique volume of recycled concrete aggregates for all types of mixtures may effectively fulfil the standard specifications of the volumetric and mechanical properties.

In another study, Ossa et al. (2016) used recycled construction and demolition waste aggregates to create asphalt mixtures. Various analyses were conducted to evaluate the moisture damage and plastic deformation of asphalt mixture. The researchers used four different percentages (10%, 20%, 30% and 40%) of waste aggregates to generate asphalt mixture specimens. The experimental results showed that up to 20% were the feasible percentage of construction and demolition waste aggregates to be used

2.3.6 Crumb Rubber

As mentioned in the previous section 2.4.1.1, the crumb rubber (CR) modified asphalt binders can be prepared using two different mixing processes, namely wet and dry processes. For the wet process, CR is blended with asphalt binder before addition into the mixture where CR is used as a modifier for asphalt binder. Meanwhile, for the dry process, the CR is used as an aggregate where it is mixed with the natural aggregate before the addition of asphalt binder into the mixture. The addition of tire rubber in the asphalt mixtures using dry process was reported to improve the resistance to permanent deformation at high temperature and cracking at low temperature (Cao, 2007). In a more recent study, Mashaan (2016a) studied the effect of waste tire rubber application on the improvement of SMA mixture mechanical properties. The results showed that the mixtures containing CR have a higher resistance to rutting. Moreover, CR of 12 - 16% by the total weight of the asphalt mixture produced the highest rutting resistance compared to the control mix (without CR).

2.4 Effect of Waste Materials as an Aggregate on SMA performance

The most critical factors in the SMA mix are the type, gradation, quality, and shape of the aggregates, which influence the internal friction between the mix components and prevent cracking (Liu et al., 2017; Devulapalli, Kothandaraman, & Sarang, 2020). The high amount of crushed and angular aggregate enhances the interlocking between the mix components and improves the mix performance (Miller, Vasconcelos, Little, & Bhasin, 2011; Liu et al., 2017). The aggregates must be of good quality to ensure good asphalt pavement performance. They must also be free from harmful materials, such as dust, dirt, and clay, that could reduce the asphalt mix performance. Asphalt mixes are designed to obtain a cost-effective mix of aggregates and asphalt binders that meet the mix design criteria. The materials must be carefully blended to ensure a good coating of the aggregates and consistency of the asphalt mix. Therefore, it is essential to choose the most appropriate aggregate, asphalt binder, and filler to prevent pavement distress (Mo, Huurman, Wu, & Molenaar, 2009).

A comprehensive search of all relevant publications that used the waste materials as an aggregate replacement in SMA during the last 11 years (2010–2020) was conducted, the extracted information was summarised t in Table 2.1. Based on the literature review, SMA samples were prepared using different compaction methods, including gyratory compactor, Marshall compactor, and roller compactor. Most research used the Marshall compactor because it is relatively light, portable, and inexpensive compared to other compactors (Kuo & Du, 2010; Behnood & Ameri, 2012; Moghaddam, Karim, & Syammaun, 2012; Rongali, Singh, Chourasiya, & Jain, 2013; Chen & Wei, 2016; Nguyen & Tran, 2018; Sangiorgi et al., 2018; Bharath, Reddy, Tandon, & Reddy, 2019; Devulapalli, Kothandaraman, & Sarang, 2019; Wang, Fan, Li, Wang, & Huang, 2019; Devulapalli et al., 2020; Huang, Qian, Hu, & Zheng, 2020; Nwakaire et al., 2020).

Table 2.1: Materials used as aggregate in SMA

Reference	Asphalt binder	Aggregate	Type of SMA	Waste Material				Performance test									
				Waste type	%, by aggregate weight	Fine	Coarse	Rutting	Stiffness	Moisture	Fatigue	Creep	Drain down	low temp.	ITS	Skid	Cantabro
(Bharath et al., 2019)	VG-30	NS	NS	RAP	25, 35, 45	✓	✓		✓		✓				✓		
(Jomoor, Fakhri, & Keymanesh, 2019)	60/70	Limestone	SMA 20	RAP	10, 20, 30, 40, 50	✓	✓					✓					
(Lo Presti, Hassan, Khan, & Airey, 2015)	PMB 25/55-55	NS	SMA 20	RAP	15, 30	✓	✓										
(Fernandes, Silva, & Oliveira, 2018)	35/50	Granite	SMA 14	RAP	50	✓	✓	✓	✓	✓	✓		✓				
(Zhu, Zhao, & Yang, 2011)	NS	Limestone & basalt	SMA 16	RAP	20, 30, 40, 50, 60	✓	✓			✓							
(Fan, Wu, & Xu, 2019)	100/120	NS	SMA 13	RAP	10, 20, 30	✓	✓	✓			✓			✓			
(Hyzl, Dasek, Coufalikova, Varaus, & Stehlik, 2019)	PMB 45/80-55	NS	SMA 11	RAP	10, 20, 30, 50	✓	✓	✓	✓					✓			
(Jagadeesh & Kadayyanvarmath, 2017)	PMB-70	NS	SMA 13	RAP	8, 16, 24	✓	✓	✓		✓			✓		✓		
(Devulapalli et al., 2019)	60/70	Granite	SMA 20	RAP	10, 20, 30, 40	✓	✓			✓			✓		✓		
(Devulapalli et al., 2020)	VG-30	Granite	SMA 13	RAP	10, 20, 30, 40	✓	✓			✓			✓		✓		
(Kuo & Du, 2010)	NS	NS	SMA 13	RCA	50, 100	✓	✓	✓	✓								
(Nwakaire et al., 2020)	80/100	Granite	SMA 14	RCA	20, 40, 60, 80, 100	✓	✓	✓	✓	✓	✓					✓	
(Pourtahmasb & Karim, 2014a)	80/100	Granite	SMA 20	RCA	20, 40, 60, 80	✓	✓	✓	✓								
(Pourtahmasb, Karim, & Shamshirband, 2015)	80/100	Granite	SMA 20	RCA	20, 40, 60, 80	✓	✓		✓								
(Pourtahmasb & Karim, 2014b)	80/100	Granite	SMA 20	RCA	20, 40, 60, 80	✓	✓										
(Chen & Wei, 2016)	AC 20	Limestone	SMA 20	Steel slag	100		✓	✓	✓	✓						✓	

Reference	Asphalt binder	Aggregate	Type of SMA	Waste Material				Performance test									
				Waste type	%, by aggregate weight	Fine	Coarse	Rutting	Stiffness	Moisture	Fatigue	Creep	Drain down	low temp.	ITS	Skid	Cantabro
(Oluwasola & Hainin, 2016; Oluwasola, Hainin, & Aziz, 2016)	80/100	Granite	SMA 14	Steel slag	40, 80	✓	✓	✓	✓	✓		✓			✓		
(Huang, 2017)	AC 20	Granite	NS	Steel slag	20, 40, 60		✓							✓			
(Groenniger, Falchetto, Isailović, Wang, & Wistuba, 2017)	50/70	Gabbro	SMA 11	Steel slag	100	✓	✓		✓		✓	✓		✓		✓	
(GROENNINGER & Wistuba, 2017)	50/70	Gabbro	SMA 11	Steel slag	100	✓	✓	✓	✓		✓			✓			
(Behnood & Ameri, 2012)	60/70	Limestone		Steel slag	100	✓	✓		✓	✓					✓		
(Chen, Wu, Pang, & Xie, 2016)	NS	Basalt	SMA 13	Steel slag	29, 45, 100	✓						✓				✓	
(Alinezhad & Sahaf, 2019)	PG 64-22	Limestone	NS	Steel slag	75		✓				✓		✓				
(Ameli et al., 2020)	60/70	Limestone	SMA 13	Steel slag	100		✓	✓	✓			✓			✓		
(Pasetto & Baldo, 2012)	60/70	Limestone	SMA 13	Steel slag	26, 59	✓	✓	✓	✓	✓	✓	✓			✓		
(Chen et al., 2015)	60/70	Gneiss	NS	Steel slag	100	✓		✓		✓							
(Jakarni, Yusoff, Wu, & Aziz, 2015)	60/70	Granite	SMA 14	Molten slag	20	✓	✓										
(Hainin, Rusbintardjo, Aziz, Hamim, & Yusoff, 2013)	PG-76	Granite	SMA 14&20	Steel slag	100	✓	✓	✓	✓			✓					
(Huang et al., 2020)	50/70	Basalt	SMA 13	Waste ceramic	10, 20, 30, 40, 50		✓					✓		✓	✓	✓	
(Moghaddam et al., 2012)	80/100	Granite	SMA 13	Waste plastic	0.2, 0.4, 0.6, 0.8, 1.0	✓			✓		✓						
(Rongali et al., 2013)	VG-30	Granite	NS	Waste plastic	8**	✓		✓	✓	✓					✓		

Reference	Asphalt binder	Aggregate	Type of SMA	Waste Material				Performance test									
				Waste type	%, by aggregate weight	Fine	Coarse	Rutting	Stiffness	Moisture	Fatigue	Creep	Drain down	low temp.	ITS	Skid	Cantabro
(Sarang, Lekha, & Shankar, 2014)	VG-30	Granite	SMA 13	Waste plastic	4, 8, 12, 16*	✓				✓			✓	✓	✓		
(Beena & Bindu, 2010)	60/70	Granite	SMA 13	Waste plastic	6, 7, 8, 9, 10, 11, 12*	✓				✓			✓	✓			
(Sarang, Lekha, Krishna, & Ravi Shankar, 2015)	VG-30	Granite	SMA 13	Waste plastic	4, 8, 12, 16*	✓		✓		✓	✓		✓	✓	✓		
(Ahmadinia et al., 2012)	80/100	Granite	SMA 20	Waste plastic	2, 4, 6, 8, 10*	✓		✓	✓	✓			✓	✓			
(Ahmadinia et al., 2011)	80/100	Granite	SMA 20	Waste plastic	2, 4, 6, 8, 10*	✓											
(Sangiorgi et al., 2018)	NS	NS	NS	CR	0.75, 1.2	✓			✓						✓	✓	✓
(Gong et al., 2019)	PG70-22	Basalt	SMA 13	CR	25, 50, 100	✓		✓			✓				✓		
(Shen, Li, & Xie, 2017)	PG 67-22	Granite	NS	CR	10*	✓											
(Nguyen & Tran, 2018)	60/70	Granite	SMA 13	CR	1, 1.5, 2, 3	✓		✓							✓		
(Mashaan et al., 2013)	80/100	Granite	SMA 14	CR	6, 12, 16, 20*	✓									✓		
(Xie & Shen, 2013)	PG 67-22	Granite	SMA 13	CR	11.4*	✓											
(Wang et al., 2019)	SK70	Granite	SMA 13	CR	3	✓								✓			✓
(Malarvizhi, Senthil, & Kamaraj, 2012)	60/70	Granite	NS	CR	15, 30*	✓				✓			✓		✓		

RAP: Recycled asphalt pavement, **RCA**: Recycled concrete aggregates, **CR**: Crumb rubber, **NS**; not specified * By binder weight, ** By filler weight

The compaction rate is dependent on the traffic condition and could be 35, 50, or 75 blows on either side of the sample. Heavily trafficked roads constructed using dense graded asphalt mixes are compacted with 75 blows. However, applying 75 blows on the SMA mixes often break the aggregates and did not significantly increase their density. Some researchers recommended applying 50 blows per side of each mix (Beena & Bindu, 2010; Kuo & Du, 2010; Ahmadinia et al., 2011; Ahmadinia et al., 2012; Behnood & Ameri, 2012; Malarvizhi et al., 2012; Hainin et al., 2013; Mashaan et al., 2013; Rongali et al., 2013; Pourtahmasb & Karim, 2014b; Chen & Wei, 2016; Devulapalli et al., 2019, 2020; Huang et al., 2020).

The optimum asphalt binder content (OBC) is dependent on the mix's stability, flow, specific gravity, and air voids. For SMA mixes, air voids are the critical determiner of OBC (National Asphalt Pavement Association, 2002; Asi, 2006; Panda, Suchismita, & Giri, 2013; Choudhary, Kumar, & Murkute, 2018). The SMA mix should be designed to have an optimum binder content that produces 4% air voids (Kuo & Du, 2010; Sarang et al., 2015; Chen & Wei, 2016; Devulapalli et al., 2019, 2020). Table 2.2 shows that the mix's optimum binder content of the control SMA is between 5.5% - 6.6%. According to the specifications adopted by India, Malaysia, and the National Asphalt Pavement Association (NAPA), the minimum optimum binder content for the SMA mix is 5.8% (Congress, 2008) and 6.0% (National Asphalt Pavement Association, 2002; Jabatan Kerja Raya Malaysia (JKR), 2008). The optimum binder for asphalt mixes containing wastes decreased with higher percentages of waste aggregates replacement (Moghaddam et al., 2012; Sarang et al., 2014; Huang, 2017; Fan et al., 2019). However, several studies showed that the required asphalt binder increase with higher waste contents (Hainin et al., 2013; Pourtahmasb & Karim, 2014a; Sangiorgi et al., 2018; Huang et al., 2020).

Table 2.2: Optimum binder content

Reference	OBC, %
(Hainin et al., 2013; Huang, 2017; Fernandes et al., 2018; Huang et al., 2020)	5.5 - 5.7
(Behnood & Ameri, 2012; Rongali et al., 2013; Xie & Shen, 2013; Chen, Wu, Pang, et al., 2016; Alinezhad & Sahaf, 2019; Nwakaire et al., 2020)	5.8 - 6.0
(Malarvizhi et al., 2012; Pourtahmasb & Karim, 2014a; Jagadeesh & Kadayyanvarmath, 2017; Jomoor et al., 2019)	6.1 - 6.3
(Beena & Bindu, 2010; Sangiorgi et al., 2018; Fan et al., 2019; Hyzl et al., 2019)	6.4 - 6.6

Fatigue and rutting are the common defects on roadway surfaces and are caused by an increase in the number of vehicles and load, environmental conditions, construction, and design errors (Moghaddam, Karim, & Abdelaziz, 2011). The following subsection will discuss the effects of using waste materials as aggregates replacement on the rutting and fatigue performance.

2.4.1 Rutting Performance

Rutting (permanent deformation) is one of the pavement distresses that affect asphalt mix performance and is caused by heavy traffic loads and hot climates. The literature described several tests for measuring pavement rutting performance, including wheel tracking, Marshall quotient, indirect tensile, and creep tests (Tayfur, Ozen, & Aksoy, 2007; Moghaddam et al., 2011). The asphalt mixes with a higher Marshall quotient value have higher stiffness and better load distribution, which contribute to higher resistance rutting. Asphalt mixes should have a Marshall quotient at least 2 kN/mm (Jabatan Kerja Raya Malaysia (JKR), 2008; MORTH, 2013).

The wheel tracking test is often used due to its good field simulation performance (Lu & Redelius, 2007). This test assesses the failure sensitivity of asphalt mixes caused by the weakness in the aggregate composition, inadequate binder coating, and poor adhesion between binder and aggregates. Traffic level, mix design, and weather conditions affect the rutting resistance of asphalt mixes. The following mix design factors affect the permanent deformation of asphalt mixes:

1. A low air voids percentage results in a low permanent deformation (Ahmadinia et al., 2011).
2. Large aggregate particle size results in a low permanent deformation (Chiu & Lu, 2007; Fakhri, Kheiry, & Mirghasemi, 2012; Hafeez, Kamal, & Mirza, 2014; Hafeez, Kamal, & Mirza, 2015).
3. Softer asphalt binders have higher permanent deformation (Fernandes et al., 2018)
4. Mixes with high asphalt contents have a high permanent deformation (Li, Ni, Li, & Wang, 2013).

Wheel tracking test can be conducted on a cylindrical or slab sample used. The slab samples yield more reliable results due to their larger size and uniform geometry compared to the cylindrical samples. Besides, cylindrical samples have to be glued together to prevent localized failure due to larger peak maximum principal strains in the absence of bonding between the cylindrical samples (Tsai, Coleri, Harvey, & Monismith, 2016). Table 2.3 summarizes the effects of recycled asphalt, recycled concrete, steel slag, palm oil clinker, plastic, and crumb rubber on the rutting performance of SMA mixes.

Table 2.3: The effect of waste materials on rutting performance

Reference	Waste	Sample shape	Condition	Temp. (°C)	Effect
(Fernandes et al., 2018)	Recycled asphalt	S	Dry	60	The performance recycled SMA mix containing modified asphalt binder is superior to the control mix.
(Fan et al., 2019)		Cylindrical	Wet	NS	The rutting of the SMA mix increased with a high RAP content before decreasing with higher RAP contents. The optimal RAP content is 20%.
(Hyzl et al., 2019)		S	Dry	60, 50	Higher RAP content did not result in a significantly higher rutting resistance of the SMA mix.
(Jagadeesh & Kadayyanvarmath, 2017)		NS	Wet	NS	The SMA mix containing an optimal amount of RAP (16%) has a higher rutting resistance than the conventional SMA mixes.

Reference	Waste	Sample shape	Condition	Temp. (°C)	Effect
(Kuo & Du, 2010)	Recycled concrete	S	Dry	60, 25	The SMA containing 50% crushed concrete showed less deformation than the other mixes.
(Nwakaire et al., 2020)		S	Dry	45	The mix containing 80% recycled concrete content has a greater rut depth. The SMA with 40% recycled concrete as coarse aggregates has the preferable rut resistance.
(Pourtahmasb & Karim, 2014a)		C	Dry	45	The SMA mixes containing 20% and 40% recycled concrete aggregates have better rutting resistance than those with 60 and 80% recycled concrete aggregates.
(Chen & Wei, 2016)	Steel slag	S	Dry	NS	The SMA mix containing steel slag has a high resistance to rutting when used as coarse aggregates due to their high angular shape and rough surface texture.
(Oluwasola & Hainin, 2016)		C	Dry	60	Using steel slag as aggregate replacement in SMA mixes increased the rutting resistance of the mixes.
(Ameli et al., 2020)		NS	NS	60	Using steel slag as coarse aggregates enhanced the aggregate interlock and the mix's rutting resistance.
(Chen et al., 2015)		C	Dry	60	The mix containing modified steel slag has better deformation resistance than the asphalt mix containing original steel slag.
(Hainin et al., 2013)		S	Dry	NS	The asphalt mixes containing steel slag have a high resistance to permanent deformation due to the excellent binding properties of the steel slag aggregate particles with the asphalt and low flakiness index.
(Rongali et al., 2013)	Plastic waste	S	NS	45	Incorporating plastic waste in the SMA mix resulted in significantly lower rutting resistance.
(Sarang et al., 2015)		S	Dry	30–35	Asphalt mix containing 8% plastic waste has better rutting resistance.
(Ahmadinia et al., 2012)		S	Dry	45	The SMA mixes containing waste plastic have lower rutting values than the control mix. The asphalt mix containing 4% waste plastic has the lowest rut depth.
(Gong et al., 2019)	Crumb rubber	C	Wet	50	Partial replacement of the mineral aggregates with crumb rubber reduced the rutting resistance of the SMA mix.
(Nguyen & Tran, 2018)		S	Dry	60	The mix with an optimal CR content of 1.5%-2.0% improved the rutting resistance of the dry process asphalt mix to equivalent to the CR modified asphalt mixes produced using the wet process.

C; Cylindrical**, S**; Slab, **NS**; not specified

Several researchers recommended an optimal content of 40% (Kuo & Du, 2010) and 50% (Pourtahmasb & Karim, 2014a; Nwakaire et al., 2020) recycled concrete aggregate. The aggregate's angular shape and rough-textured surface are critical in determining the deformation resistance of the SMA at high temperatures. Recycled concrete aggregates have a more angular shape and rougher particles than the normal aggregate, and thus have higher internal friction that increased the rutting resistance (Kuo & Du, 2010).

The SMA mixes incorporated with steel slag have a better rutting resistance than the control mix. Steel slag can be used as a 100% replacement of the aggregates because of its high specific gravity and Los Angeles abrasion. The steel slag also has a significantly more angular shape and rougher surface texture than the conventional aggregates, which improved the aggregate interlock and binding properties, and thus increased the mixture's rutting resistance.

Plastics are typically organic polymers with a high molecular mass and often contain other substances. They are generally characterized as water-insoluble. The plastic particles were incorporated into the asphalt mix using the dry process to retain the natural state of the plastic particles and causing minimal change to their shape and properties (Ahmadinia et al., 2012). Because some plastics have a higher melting point than the maximum temperature for the blending, researchers recommended using plastics in hot-mix asphalts as an aggregate replacer (dry process) instead of using an asphalt binder modifier. The incorporation of waste plastics as a fine aggregate replacement in the asphalt mix improved the mix's rutting resistance primarily because the higher melting point of the plastic particles allows them to remain as a semi-crystalline substance in the mix and thus produce a stiffer mix (Ahmadinia et al., 2012).

2.4.2 Fatigue Performance

Asphalt pavements are exposed to repetitive loads and varying temperatures in the long term, which cause the formation of internal micro-cracks due to the accumulation of tensile stress. As more micro-cracks formed and spread on the pavement surface, they eventually cause the pavements to collapse.

The lifespan of asphalt pavements is dependent on its fatigue performance, where asphalt mixes with better fatigue performance have a longer lifespan. Asphalt mixes containing fragile materials are more likely to have a shorter lifespan. The fatigue life of an asphalt mix is dependent on several factors, including the content and grade of the asphalt binder, aggregate gradation, air voids, age of the asphalt mix, and pavement layer thickness (Moghaddam et al., 2012; Bharath et al., 2019). The literature reported employing different equipment, specimen geometry, and loading configurations to perform the fatigue test. The fatigue performance of an asphalt mix can be determined using tension-compression fatigue, trapezoidal cantilever fatigue (or trapezoidal fatigue), three-point flexural, four-point bending beam, and diametral loading tests (or indirect tensile test) under controlled stress or controlled displacement (Di Benedetto, De La Roche, Baaj, Pronk, & Lundström, 2004; Moghaddam et al., 2011). The test can be performed on a cylindrical (100mm diameter and 64 $\pm$ 2mm thickness) or beam (380 x 50 x 63 mm) sample. Several studies performed the indirect tensile fatigue test to determine the fatigue performance of the asphalt mixes (Moghaddam et al., 2012; Pasetto & Baldo, 2012; Sarang et al., 2015; Groenniger et al., 2017; GROENNINGER & Wistuba, 2017; Bharath et al., 2019; Fan et al., 2019; Gong et al., 2019; Nwakaire et al., 2020) because this test requires the use of less complex testing equipment and is less time-consuming. Generally, the incorporation of waste materials such as recycled asphalt, recycled concrete, steel slag, palm oil clinker, waste plastic, and crumb rubber as an aggregate replacement improves the fatigue cracking resistance and extends the fatigue life of the mixtures. The

SMA mix has a longer fatigue life than the dense-graded mix because of its higher asphalt binder content (Bharath et al., 2019). Table 2.4 summarizes the effects of these waste materials on fatigue performance.

Table 2.4: The effects of wastes on the fatigue performance

Reference	Waste	Temp. (°C)	Load shape	Mode	Test	Effect
(Bharath et al., 2019)	Recycled asphalt	25	Sinusoidal	Control stress	ITFT	The incorporation of higher percentages of recycled asphalt resulted in a significantly shorter fatigue life.
(Fernandes et al., 2018)	Recycled asphalt	20	Sinusoidal	NS	FPBT	The recycled asphalt produced from waste engine oil and polymers has the best fatigue cracking resistance, while the recycled SMA mixes containing crumb rubber have the worst behaviour. Waste engine oil has a positive influence on fatigue cracking resistance.
(Fan et al., 2019)	Recycled asphalt	15	NS	Control strain	ITFT	SMA mixes containing recycled asphalt have higher fatigue life values than the control mix. The optimal percentage for recycled asphalt is 20%.
(Nwakaire et al., 2020)	Recycled concrete	25	Sinusoidal	Control stress	ITFT	Higher percentages of recycled concrete resulted in longer fracture life due to the higher amount of binder in the matrix. The asphalt mixes containing recycled concrete are much more brittle than the control mix at the same binder content.
(Groenniger et al., 2017) (GROENNINGER & Wistuba, 2017)	Steel slag	20,10,0, and -10	Sinusoidal	NS	ITFT	The incorporation of steel slag into the SMA mix resulted in a higher fatigue resistance than mixes containing natural gabbro aggregates.
(Alinezhad & Sahaf, 2019)	Steel slag	20	NS	NS	FPBT	The SMA mix containing steel slag as coarse aggregate replacement has a higher fatigue resistance due to the better interlocking of the steel slag and higher angle of friction of steel slag than the normal aggregate.
(Pasetto & Baldo, 2012)	Steel slag	20	Sinusoidal	Control stress	ITFT	The asphalt mixes containing steel slag have a higher fatigue resistance than the asphalt mix containing natural aggregates.
(Moghaddam et al., 2012)	waste plastic	20	Haversine	Control stress	ITFT	The waste plastic in the SMA mix significantly improved the mix's fatigue properties, and asphalt mixes with higher waste plastic contents have longer fatigue lives.
(Sarang et al., 2015)	waste plastic	25	Half sine	NS	ITFT	Higher waste plastic content resulted in considerably improved fatigue life.

Reference	Waste	Temp. (°C)	Load shape	Mode	Test	Effect
(Gong et al., 2019)	Crumb rubber	25	NS	NS	FPBT	The SMA mixes containing pre-coated rubber aggregates have better fatigue cracking resistance than those containing untreated rubber aggregates. At the same percentage of replacement, the asphalt mix with a bigger crumb rubber has lower fatigue resistance performance. The mix with a higher replacement percentage of crumb rubber of the same size has a better fatigue resistance performance.

2.5 Palm Oil Industry

Palm oil is the main contributor to world vegetable oil market, sharing up to 36% of the market with 70.7 million tonnes of production reported in 2018. Figure 2.4 shows the world vegetable oil consumption in 2018 (American Soybean Association, 2018).

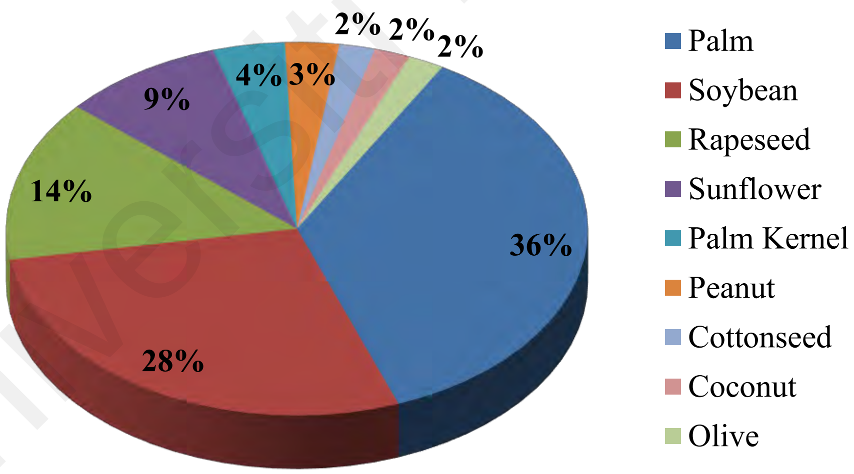


Figure 2.4: World vegetable oil consumption in 2018 (American Soybean Association, 2018)

Malaysia is one of the main producers of palm oil products in the world and is the second-largest palm oil producer in the world with the ability to generate up to 32% of total palm oil production (Lam et al., 2019). It has produced 19.86 million tonnes of palm oil in 2019 (Malaysian Palm Oil Board, 2019). Table 2.5 displays palm oil production for different states in Malaysia as of the year 2019. In 2019, 446 palm oil mills and 51 palm oil refineries were reported to have been operating in Malaysia.

Table 2.5: Palm oil production (tonnes) in Malaysia (Malaysian Palm Oil Board, 2019)

State	**Palm oil production (tonnes)**
Johor	3,142,218
Kedah	239,073
Kelantan	332,438
Negeri Sembilan	698,440
Pahang	3,019,773
Perak	1,854,050
Selangor	521,837
Terengganu	556,230
Sabah	5,037,168
Sarawak	4,237,411
Other States	219,729
Malaysia (Total production)	**19,858,367**

Oil palm tree plantation in Malaysia increase continuously in line with the Malaysian government's strategies for the production of palm oil-based biodiesel (Hosseini & Wahid, 2012). Malaysian Palm Oil Board (MPOB) reported that the land area committed to oil palm plantation in 2019 accounted for about 5,849,330 hectares. Table 2.6 depicts oil palm plantation areas in different states of Malaysia as of December 2018 (Malaysian Palm Oil Board, 2018). The palm oil industry plays an important role in the economic development of Malaysia and has been a key contributor to the gross domestic product (GDP) with an estimation of RM 80 billion in 2018, making up to 8.7% of Malaysia's total GDP (Andiappan et al., 2019).

Table 2.6: Oil palm plantation area in Malaysia (Malaysian Palm Oil Board, 2018)

State	Matured (hectares)	%	Immature (hectares)	%	Total (hectares)	%
Johor	680,562	91.0	67,000	9.0	747,562	12.8
Kedah	82,287	91.1	8,007	8.9	90,294	1.5
Kelantan	121,085	77.9	34,287	22.1	155,372	2.7
Melaka	51,237	90.2	5,574	9.8	56,811	1.0
Negeri Sembilan	167,026	89.1	20,425	10.9	187,451	3.2
Pahang	653,535	86.4	102,614	13.6	756,149	12.9
Perak	364,090	88.1	49,221	11.9	413,311	7.1
Perlis	641	94.1	40	5.9	681	0.01
Pulau Pinang	14,042	95.5	660	4.5	14,702	0.3
Selangor	123,139	90.3	13,222	9.7	136,361	2.3
Terengganu	149,519	88.5	19,395	11.5	168,914	2.9
Sabah	1,378,655	89.0	170,590	11.0	1,549,245	26.5
Sarawak	1,403,526	89.3	168,951	10.7	1,572,477	26.9
Malaysia (Total)	**5,189,344**	**88.7**	**659,986**	**11.3**	**5,849,330**	**100**

2.5.1 Oil Palm Waste

Generally, oil palm trees (Figure 2.5) are planted for food applications (cooking oil and margarine) and non-food applications (animal feed, biodiesel and energy generation). Palm trees have a vertical trunk and feathery leaves known as palm frond. Bunches of palm fruit develop between the trunk and base of new fronds. After 5 to 6 years of planting the oil palm tree, the first fresh fruit crop can be harvested and each tree can supply palm fruit for 25 to 30 years with each fresh fruit bunch weighing around 10 to 40 kg. (Hosseini & Wahid, 2014).

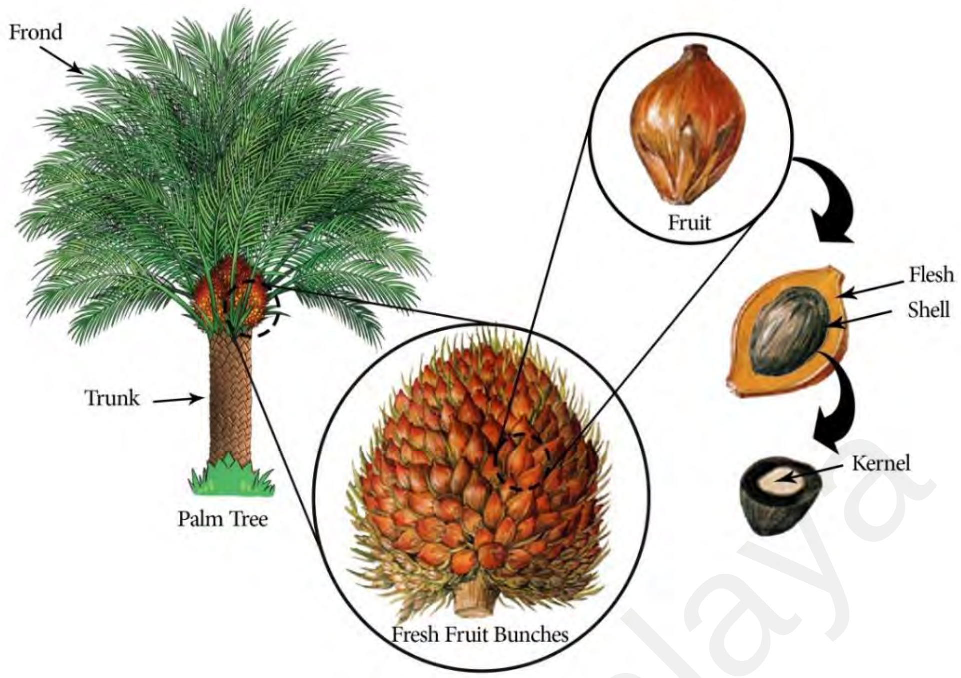


Figure 2.5: Oil palm tree

Palm oil processing activities results in only 10% of oil from the fresh fruit and kernel while the remaining 90% remain in the form of biomass or waste which is underutilised (Dungani et al., 2018). In Malaysia, palm oil industries generate various types of waste materials such as empty fruit bunches, palm oil mill effluents, fronds, trunks, fibres and kernel shells. Indirectly, the palm oil industry increases the pollution issue where millions of tonnes of wastes were produced annually (Mohammed et al., 2011). These wastes have been projected to increase parallel with the ongoing global consumption demand for palm oil.

2.5.1.1 Palm Oil Clinker and Fuel Ash

Oil palm shells and fibres are burned together in mills to generate energy because they have high fuel value resulting in new waste materials such as palm oil clinker (POC) and palm oil fuel ash (POFA). POC is a by-product produced in a large amount as a result of palm oil processing which is like a porous stone, grey and has irregular and flaky shape (Ahmad, Hilton, & Mohd Noor, 2007). The POFA varies in the degree of colour from grey to darker shade based on the carbon content. The darker shade can be attributed to the high amount of unburned carbon. Moreover, POFA constitutes about 5% of total waste materials after burning the oil palm fruit shells and fibres to generate electricity in palm oil mills (Tangchirapat, Saeting, Jaturapitakkul, Kiattikomol, & Siripanichgorn, 2007). In Malaysia, oil palm wastes of commercial value can be converted into potential construction materials such as an aggregate or cement replacer with engineering potentials and economic advantages. In general, lightweight materials such as aggregate can reduce a dead load of concrete structures and keep the strength of concrete at a suitable level which can be achieved by using POC. Several studies have utilised POC as an aggregate replacer in concrete construction (Kanadasan et al., 2015; Ibrahim & Razak, 2016; Ibrahim et al., 2017; Nayaka et al., 2019).

POFA in concrete structures can improve concrete properties by resisting chloride attack (Chindaprasirt et al., 2008), increasing the drying shrinkage (Tangchirapat & Jaturapitakkul, 2010), resisting sulphate attack (Jaturapitakkul et al., 2007), and acting as a cement replacer (Alsubari et al., 2014; Khankhaje et al., 2018; Arif et al., 2019; Hosen et al., 2019). Besides, POFA has been also reported to improve the properties of asphalt mixture used in highway construction by functioning as a filler replacer (Kamaluddin, 2008; Borhan et al., 2010; Ahmad et al., 2012; Maleka et al., 2014), and partial replacer of asphalt binder (Rahman, Hainin, & Bakar, 2017).

A) Physical and chemical properties of POC and POFA

Physical properties of POC have been reported by many researchers as presented in Table 2.7. Specific gravity can be defined as the ratio between material density and water density. It can be seen that POC possess varying specific gravity values with a range from 2.01 to 2.15 for fine POC aggregate, and from 1.5 to 1.88 for coarse POC aggregate. The difference in specific gravity can be explained by the porous structure of POC aggregate which enables higher water absorption as compared to natural aggregate (Mohammed et al., 2014). As shown in Table 2.7, the 24-hour water absorption value of POC ranged from 5.75% to 26.5%, and from 3.0% to 5.7% for fine and coarse aggregates of POC, respectively. Meanwhile, the chemical composition of POC powder is as summarised in Table 2.8.

Table 2.7: Physical properties of POC

Reference	Aggregate type	Aggregate size (mm)	Specific gravity	Water absorption (%)	Moisture content (%)
Nayaka et al. (2019)	Fine	–	–	–	–
	Coarse	5-10	1.51	5.5	0.31
Hamada, Yahaya, Muthusamy, Jokhio, and Humada (2019)	Fine	–	–	–	–
	Coarse	4.75-10	1.78	5.7	0.38
Abutaha, Razak, Ibrahim, and Ghayeb (2018)	Fine	< 4.75	2.15	5.75	0.11
	Coarse	4.75-10	1.81	4.35	0.28
Ibrahim et al. (2017)	Fine	–	–	–	–
	Coarse	4.75-9.5	1.88	3±2	–
Abutaha, Abdul Razak, and Kanadasan (2016)	Fine	< 5	2.15	10±5	0.5±0.25
	Coarse	5-14	1.73	3±2	1±0.5
Ahmmad et al. (2016)	Fine	–	–	–	–
	Coarse	–	1.75	5.67	0.08
Kanadasan and Razak (2014)	Fine	< 5	2.15	10±5	0.5±0.25
	Coarse	5-10	1.73	3±2	1±0.5
Mohammed, Foo, and Abdullahi (2014)	Fine	< 5	2.01	26.45	0.11
	Coarse	5-14	1.82	4.35	0.07

POC is like a porous stone, grey and has irregular and flaky shape with a rough surface texture (Ahmad et al., 2007). It is considered a brittle material as it has Los Angeles abrasion around 49%, and crushing value 46-56%. (Nayaka et al., 2019)(Abutaha et al., 2018; Hamada et al., 2019). In pavement construction, to ensure good asphalt pavement performance the aggregates must be of good quality and meet the standered requirements, this makes the possibility of using POC as coarse aggtregates in asphalt mixture almost non-existent due to the low strenght. Moreover, since it is a porous material and has large voids, it is likely that the mixture needs higher asphalt binder, and also volumetric properties criteria of mix design can not be satisfied such as; Air Voids (AV), Voids in Mineral Aggregate (VMA), and Voids Filled in Asphalt (VFA). POC is considered a brittle material, so it be easily crushed and can be utilised as potential alternative fine aggregate in asphalt mixture.

Table 2.8: Chemical composition of POC powder

Reference	Silicon dioxide (SiO_2)	Aluminum oxide (Al_2O_3)	Iron oxide (Fe_2O_3)	Calcium oxide (CaO)	Magnesium oxide (MgO)	Sodium oxide (Na_2O)	Potassium oxide (K_2O)	Sulfur trioxide (SO_3)	Loss on ignition (LOI)
Nayaka, Alengaram, Jumaat, Yusoff, and Alnahhal (2018); Darvish, Alengaram, Poh, Ibrahim, and Yusoff (2020)	60.29	5.83	4.71	3.28	4.2	0.20	7.24	0.31	5.23
Karim, Chowdhury, Zabed, and Saidur (2018)	65.30	4.23	5.65	3.89	3.72	–	13.65	0.09	2.42
Karim, Hashim, Razak, and Yusoff (2017)	63.9	3.89	3.30	6.93	3.37	–	10.20	0.21	–
Karim, Hashim, and Razak (2016)	62.78	3.41	6.49	6.89	3.52	0.39	10.54	0.08	3.67
Karim, Hashim, and Abdul Razak (2016)	60.29	5.83	4.71	3.27	3.76	–	7.79	0.11	–
Kanadasan and Abdul Razak (2015)	59.9	5.37	6.93	6.37	3.13	0.24	15.10	2.60	–
Kanadasan and Razak (2014)	59.9	3.89	6.93	6.37	3.3	–	15.10	0.39	1.89

Physical and chemical properties of POFA were investigated by many researchers and the findings are summarised in Table 2.9. A large variation in results was noted that could be ascribed to different conditions such as burning temperature, quantity of oil palm parts to produce POFA obtained from different factories (Thomas, Kumar, & Arel, 2017; Hamada, Jokhio, Yahaya, Humada, & Gul, 2018). The specific gravity values of POFA ranged from 2.9 to 2.04.

Table 2.9: Physical and chemical properties of palm oil fuel ash (POFA)

Reference	Specific gravity	Surface area (m^2/g)	Silicon dioxide (SiO_2)	Aluminium oxide (Al_2O_3)	Iron oxide (Fe_2O_3)	Calcium oxide (CaO)	Magnesium oxide (MgO)	Sodium oxide (Na_2O)	Potassium oxide (K_2O)	Sulfur trioxide (SO_3)	Loss on ignition (LOI)
Hamada et al. (2019)	2.52	1.962	67.30	4.12	8.12	3.97	2.72	0.115	8.45	0.535	–
Alsubari, Shafigh, and Jumaat (2016)	2.04	4.9	59.17	3.73	6.33	5.80	4.87	0.180	8.25	0.72	16.1
Liu, Alengaram, Santhanam, Jumaat, and Mo (2016)	–	–	63.40	5.5	4.20	4.30	3.70	–	6.30	0.90	6.0
Islam, Mo, Alengaram, and Jumaat (2016)	2.15	–	71.67	0.94	2.77	5.61	4.91	0.120	7.89	1.05	–
Zeyad, Johari, Tayeh, and Yusuf (2016)	2.42	43.25	51.18	4.61	3.42	6.93	4.02	0.056	5.52	0.36	15.30
Bashar et al. (2016)	2.2	1.720	67.72	3.71	4.71	5.57	4.04	0.160	7.67	1.07	6.20
Lim et al. (2015)	2.42	–	53.5	1.9	1.1	8.3	4.10	–	6.5	2.36	20.9
Awal and Shehu (2015)	2.42	–	62.60	4.65	8.12	5.70	3.52	–	9.05	1.16	6.25
Islam, Alengaram, Jumaat, and Bashar (2014)	2.20	172	63.41	5.55	4.19	4.34	3.74	0.160	6.33	0.91	6.20
Yusuf, Megat Johari, Ahmad, and Maslehuddin (2014)	2.9	14.92	46.0	3.1	2.45	8.46	4.4	0.13	4.08	0.3	21.6

2.6 Summary

Various previous studies related to the present study were reviewed and presented in this chapter. Details of hot mix asphalt method, asphalt mixture components, waste materials used in pavement constructions, and waste materials application as a modifier, aggregate, and filler were elaborated.

Besides, improved properties of asphalt binder after modification using waste materials such as plastics and crumb rubber were also discussed. Furthermore, waste materials such as glasses, steel slag, and construction demolition that were used as an aggregate and filler replacer alternative to original aggregates and fillers were explained. Most reported studies showed a positive effect of waste material substitution on the properties of asphalt mixtures.

A large volume of POC is generated in Malaysian palm oil mills as waste with an insignificant return. Hence, this industrial waste can be converted into a potential substitute in asphalt pavement construction where numerous studies have highlighted the use of POC in concrete materials construction. However, to date, the POC application in asphalt pavement construction has not been investigated. Besides, the use of POC as an aggregate in asphalt mixture design has not been reported as confirmed by systematic literature search performed. Therefore, the research gap was identified and the present study was designed to investigate the substitution of POC as a fine aggregate in asphalt pavement.

CHAPTER 3: MATERIALS AND METHODS

3.1 Introduction

This chapter focuses on experimental steps and testing methods. The chapter is divided into three main parts which are: (i) research framework, (ii) physical characterisation of materials used in this study and analysis of aggregates and asphalt binder, and (iii) information about Marshall mix design and the mechanical analyses of SMA samples.

3.2 Research Work Flowchart

An overview of four different stages of experimental work is illustrated in Figure 3.1. In the first phase, different materials such as asphalt binder, aggregate, filler and waste materials were prepared. The waste materials included palm oil clinker (POC) and palm oil fuel ash (POFA). Meanwhile, in the second phase, physical tests were conducted for all the collected materials, whereas in the third phase, SMA mixtures were formulated using Marshall's design process. Fine granite aggregates (passing sieve size 4.75 mm) were replaced with POC. Six groups of asphalt mixtures were prepared as shown in Table 3.1. The group that has 0% of waste materials is the control mixture. For each group, three duplicates samples were prepared with different asphalt binder contents (5.0%, 5.5%, 6.0%, 6.5% and 7.0%). The Marshall properties analysis and selection of optimum binder content was performed at the third stage. For the final phase, optimum binder content was used to produce other SMA samples. Then, asphalt mixture performances were determined using tests such as resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability and indirect tensile fatigue. The number of compacted samples and tests performed during stages 3 and 4 are presented in Table 3.2.

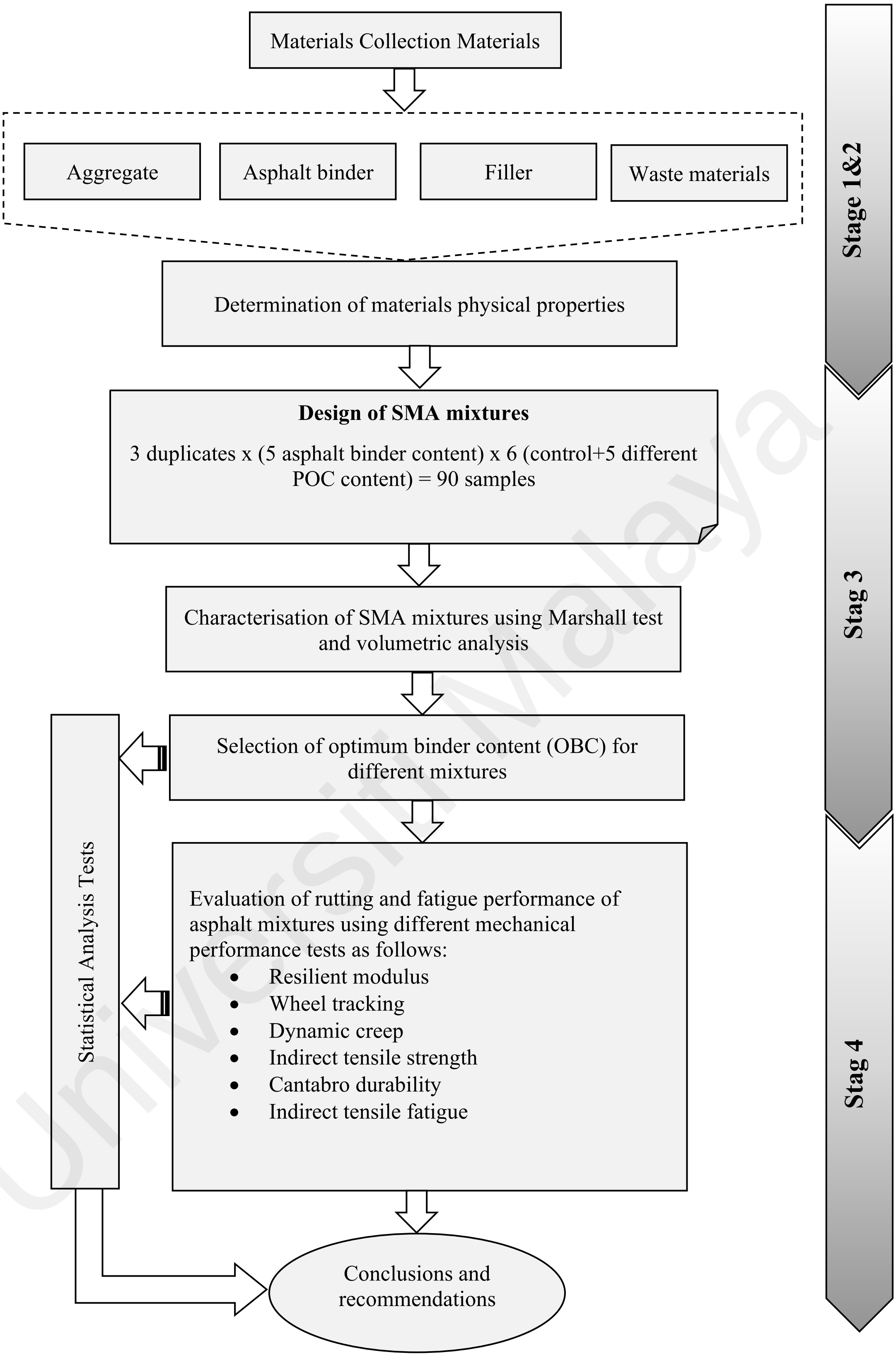


Figure 3.1: Research flowchart of a laboratory testing program

Table 3.1: Contents of different asphalt mixtures

Mixture group	Aggregates		Filler
	Coarse	**Fine**	
Control mix	100% granite	100% granite + 00% POC	100% dust + 00% fuel ash
POC-20	100% granite	80% granite + 20% POC	50% dust + 50% fuel ash
POC-40	100% granite	60% granite + 40% POC	50% dust + 50% fuel ash
POC-60	100% granite	40% granite + 60% POC	50% dust + 50% fuel ash
POC-80	100% granite	20% granite + 80% POC	50% dust + 50% fuel ash
POC-100	100% granite	00% granite + 100% POC	50% dust + 50% fuel ash

Table 3.2: The tests performed and number of samples produced

Stage	Test	Details	No. of samples	Type/ Size
Stage3: Marshall mix design	• Marshall Stability • Marshall Flow	*3 duplicates × (5 asphalt binder content) × 6 (control+5 different POC content)*	90	Cylinder 101 mm x 65 mm
Stage4: Performance of asphalt mixture	• Resilient modulus • Indirect tensile fatigue	*3 duplicate samples × 1 asphalt binder content × 6 POC contents*	18	Cylinder 101 mm x 65 mm
	• Abrasion	*3 duplicate samples × 1 asphalt binder content × 6 POC contents*	18	Cylinder 101 mm x 65 mm
	• Dynamic creep	*3 duplicate samples × 1 asphalt binder content × 6 POC contents*	18	Cylinder 101 mm x 50 mm
	• Indirect tensile strength • Moisture sensitivity	*3 duplicate samples × 1 asphalt binder content × 6 POC contents × 2 conditions dry and wet*	36	Cylinder 101 mm x 65 mm
	• Wheel tracking	*2 duplicate samples × 1 asphalt binder content × 6 POC contents × 2 conditions*	24	Slab 200 mm x 300 mm x 50mm

3.3 Materials

The materials used for this study are presented in subsections 3.3.1 to 3.3.3.

3.3.1 Asphalt Binder

In this study, 80/100 penetration grade bitumen was used as the binder. To maintain the quality of binder during binder testing preparations, the 80/100 binder was transferred from drums to one-litre containers according to ASTM D140 standard procedure (ASTM, 2015c).

3.3.2 Aggregate

Crushed granite aggregates that were required for this study were purchased from a local aggregate supplier, i.e., Kajang Rock as shown in Figure 3.2. For experimental work, aggregates were stored in large quantities in different piles according to the size of aggregates. The aggregates were stored in a storage facility located in the University of Malaya. The dried aggregates were sieved through different mesh sizes of 19.0, 12.5, 9.5, 4.75, 2.36, 0.60, 0.30, and 0.075 mm. The aggregates were shaken for 10 minutes on a sieve shaker. The aggregates that retained on each sieve were collected in separate containers according to aggregate size. The aggregate gradation of SMA20 was selected based on the Malaysian specifications of the Public Works Department for road works (Jabatan Kerja Raya (JKR) Malaysia, 2008) as shown in Table 3.3 and Figure 3.3.

Figure 3.2: Aggregates stockpile

Table 3.3: Gradation limits of combined aggregates

Sieve size (mm)	% Aggregates passing			% Aggregates retained	Sample weight (g)
	Min.	Max.	Mid.		
19.0	100	100	100.0	0.0	0.0
12.5	85	95	90.0	10.0	110
9.5	65	75	70.0	20.0	220
4.75	20	28	24.0	46.0	506
2.36	16	24	20.0	4.0	44
0.60	12	16	14.0	6.0	66
0.30	12	15	13.5	0.5	5.5
0.075	8	10	9.0	4.5	49.5
Pan				9.0	99
				100	**1100**

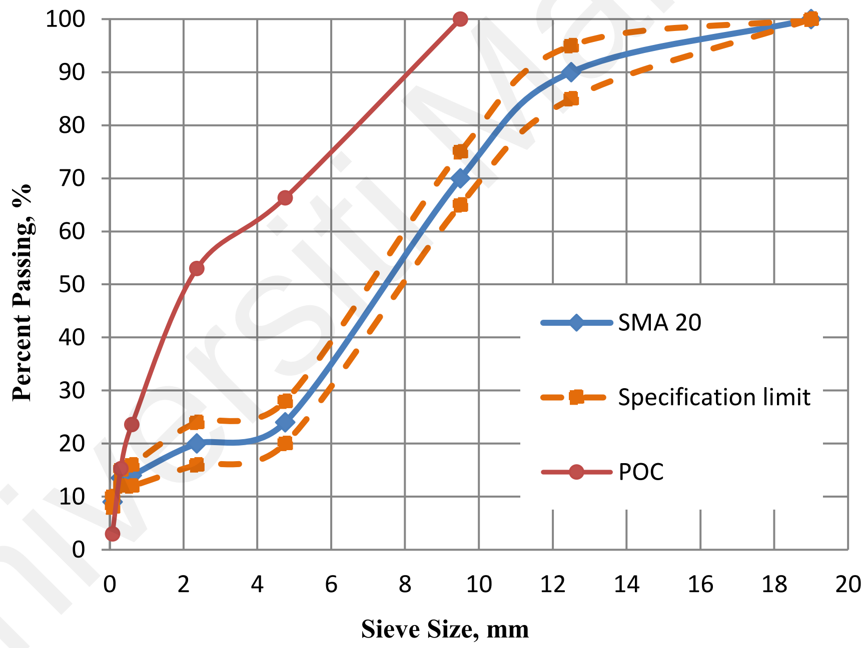


Figure 3.3: SMA20 aggregate gradation

3.3.3 Palm Oil Fuel Ash (POFA) and Clinker (POC)

POFA and POC were obtained from a local factory that processes palm oil. POC was collected in large chunks as shown in Figure 3.4 (a) and was crushed into fine aggregates as shown in Figure 3.4 (b). The crushed aggregates were then sieved and transferred into a separate container according to size. Meanwhile, POFA was collected in powder form and sieved through a sieve of mesh size 0.075 mm (#200).

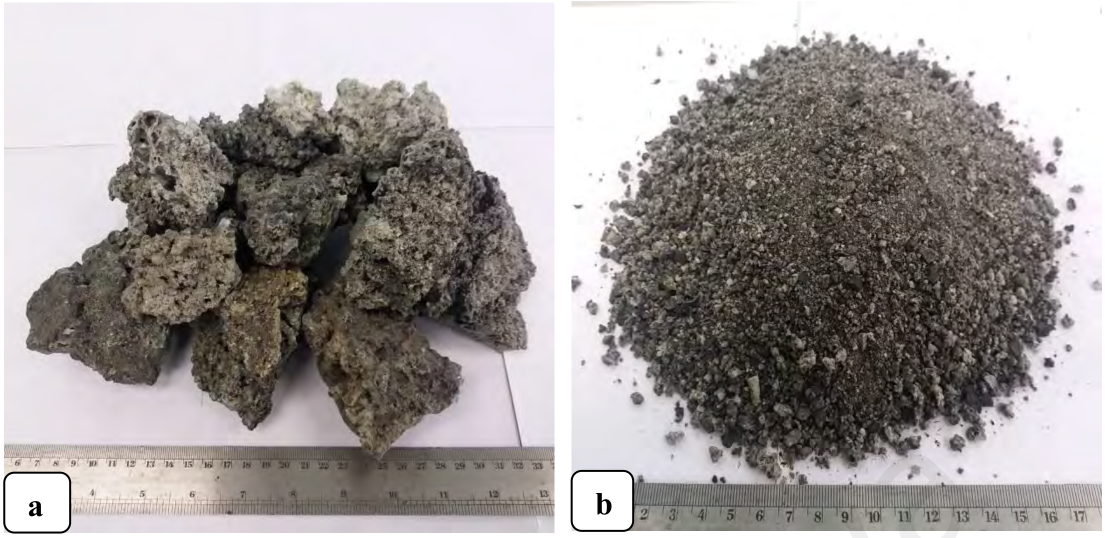


Figure 3.4: Palm oil clinker: (a) large chunks (b) crushed aggregates

Figure 3.5 shows that the POC fine aggregates contain fibres. Generally, fibres able to to reinforce the SMA mixtures and assist in generating a three-dimensional network which could improve the mix adhesion. Based on the literature review, many different types of plant-derived fibres were incorporated in the SMA to improve the mixture mechanical properties such as Posidonia oceanica (Herráiz, Herráiz, Domingo, & Domingo, 2016), coconut fibres (Awanti, Habbal, Hiremath, Tadibidi, & Hallale, 2012; Oda, Fernandes Jr, & Ildefonso, 2012; Panda et al., 2013; Prasad & Venkatesh, 2018), sisal leaves (Oda et al., 2012; Akhil, Ramu, & kalyan shetty, 2019; Kar, Giri, & Panda, 2019; Kiran Kumar & Ravitheja, 2019), jute plant fibres (Sharma & Goyal, 2006), date palm fibres (Hassan & Al-Jabri, 2005; Taallah & Guettala, 2016), oil palm fibres (Muniandy & Huat, 2006; Muniandy, Binti Che Md Akhir, Hassim, & Moazami, 2014; Syammaun & Rani, 2018; Fauzi et al., 2020), kenaf fibres (Pirmohammad, Majd Shokorlou, & Amani, 2020; Syafiqah, Masri, Jasni, & Hasan, 2021), bagasse fibres (Mansor, Zainuddin, Aziz, Razali, & Joohari, 2018; Li et al., 2020), bamboo fibres (Sheng et al., 2019; Masri, Fatin, Chin, Syafiqah, & Shaffie, 2021; Yu et al., 2021), banana fibres (Prasad & Venkatesh, 2018; Kiran Kumar & Ravitheja, 2019; Ferreira da Costa, Grangeiro de Barros, Lucena, & Elísio de Figueirêdo Lopes Lucena, 2020), corn stalk fibres (Chen, Chen, Yi, & Feng, 2021).

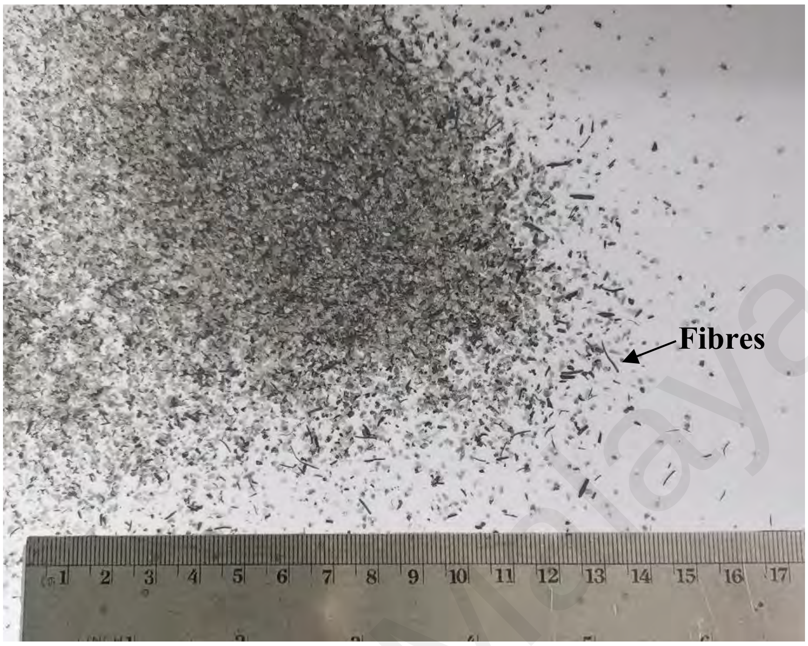


Figure 3.5: POC fine aggregates

3.4 Physical Properties of Asphalt Binder

3.4.1 Softening Point Analysis

Softening point (ring and ball) test is an empirical test that measures the temperature of asphalt when it starts to become soft and can no longer support the weight of a metal ball and begins to flow. The asphalt binder samples were heated until fluid can be poured into two brass rings and maintained at room temperature (25°C) for at least 30 minutes before the test. The rings and other parts were placed in a water bath with a depth of 105 ± 3 mm. A 9.5 mm steel ball-bearing weighing about 3.50 ± 0.05 g was centred on each specimen and temperature was raised by a rate of 5 ± 0.5 °C per minute. Finally, the mean temperature at which the two-asphalt binder samples fall at a distance of 25 mm and touches the base plate was recorded. The detailed test procedure for softening point determination is as outlined in ASTM D0036-95 (ASTM, 2000).

3.4.2 Binder Penetration Determination

A penetration test is an empirical test which measures binder consistency at intermediate service temperatures. The value of penetration depends on the hardness level of binders, where softer binder results in a higher penetration value. The penetration is defined as the distance in tenths of a millimetre (e.g., if a needle penetrates for 6 mm, the asphalt penetration will be 60). A standard needle penetrates a sample under the given condition of loading 100 grams for 5 seconds at a temperature of 25 °C. The detailed procedure of binder penetration test is described in ASTM D5 standard procedure (ASTM, 2013).

3.4.3 Viscosity Measurement

Viscosity was determined for un-aged asphalt binders to determine their flow characteristics to ensure that the binders can be pumped and handled at the hot mixing facility. A Brookfield viscometer which consists of a motor, spindle, control keys and a digital readout as shown in Figure 3.6 was used to determine the viscosity. The viscosity was reported by calculating the torque required to sustain a steady rotational speed of a cylindrical spindle at a constant temperature. The test was performed at temperatures of 135°C and 165°C.

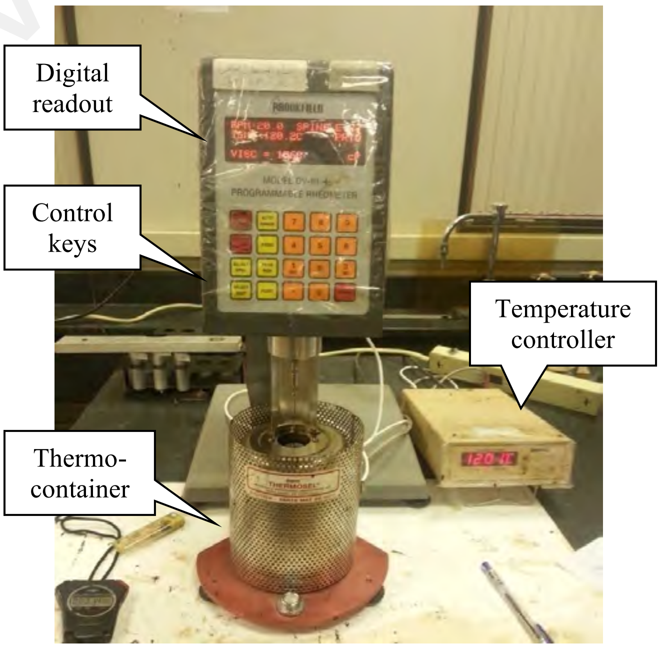


Figure 3.6: Brookfield rotational viscometer

To prepare samples for viscosity testing, samples were heated until they become fluid to be easily poured and stirred to remove entrapped air and to ensure homogeneity. About 8 to 11 grams of the binder is typically used. The pre-heated binder was placed into a sample chamber and a spindle was placed into the sample and the speed was specified at 20 rpm. The binder is ready to be tested when temperature stabilises, where approximately thirty minutes are required for the sample to reach the test temperature. The viscosity values were collected after 10 minutes of spindle rotation. Viscosity in the units of centipoises (cps) was obtained when rotational viscometer was used. To convert the units to Pascal seconds (Pa×S), the following equation was used:

$$1000\ cp = 1\ Pa \times S \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

According to the SUPERPAVE™ specification (AASHTO M320), the maximum viscosity of the asphalt binder must be less than 3000 centipoises (3 Pa ×s) at 135 ºC for storage and pumping during the construction period.

3.4.4 Dynamic Shear Rheological Analysis

Rheological properties at intermediate-temperature and high-temperature were analysed using a dynamic shear rheometer (DSR) Bohlin rheometer as shown in Figure 3.7. The DSR was used to characterise both viscous and elastic behaviours of asphalt binder by measuring the complex shear modulus (G*) and phase angle (δ). Complex shear modulus is a measure of the total resistance of a material to deformation. The phase angle is an indicator of the relative amount of recoverable and non-recoverable deformations.

The viscoelastic measurement using a DSR was carried out according to ASTM D7175 standard procedure (ASTM, 2015a). To prepare samples for testing, the asphalt was heated until it turned into a liquid and was then poured into a mould. Then the samples were placed between two 25 mm parallel plates with a gap of 1 mm for high testing temperature with the range from 58 °C to 82 °C. According to the SUPERPAVE™ specification, permanent deformation (rutting) reduction is governed by the parameter G*/sinδ. Therefore, G*/sinδ value must be greater than or equal to 1.00 kPa for un-aged asphalt binder while value should be 2.20 kPa for RTFO aged asphalt binder. High values of G* and low values of δ are desirable. Meanwhile, fatigue cracking is governed by the parameter (G*.sinδ). The G*.sinδ is known as the fatigue cracking factor which should be less than 5000 kPa for RTFO/PAV aged asphalt binder. Low values of G*.sinδ are desirable to resist fatigue cracking.

Figure 3.7: DSR (Bohlin rheometer)

3.5 Physical Properties of Aggregates

3.5.1 Aggregate Impact Value Test

Aggregate impact value test was conducted to determine coarse aggregate impact value according to British Standard (BS 812 Part 3). The sample should pass through a sieve of mesh size 12.5 mm and should be retained by a sieve of mesh size 9.5 mm. The coarse aggregate sample was then placed into a steel mould and was subjected to 15 blows of a heavy hammer weighing about 14 kg that fall from a height of 38 cm which was then removed from the mould and sieved through a sieve of mesh size 2.36 mm. Finally, the mass of passing through a sieve of mesh size 2.36 mm was weighed and expressed as a percentage of the total mass of aggregates. The suitable value of aggregate impact on asphalt pavement is 15% or less. Figure 3.8 shows the aggregate impact apparatus to determine the aggregate impact value.

Figure 3.8: Aggregate impact test apparatus

3.5.2 Los Angeles Abrasion Test

The aggregates used to construct the top layer of highway pavement should be of high quality or less of abrasion loss to withstand high stress due to heavy traffic. The pavement surface is exposed to abrasion and thus requires hard and tough materials. Los Angeles Abrasion test is one of the common tests used to evaluate the toughness and abrasion resistance of aggregates. This test was carried out according to ASTM C 131 standard procedure (ASTM, 2014) using Los Angeles abrasion machine as shown in Figure 3.9. Based on the ASTM, the abrasion value of coarse aggregate should not exceed 30%. The coarse aggregate sample was prepared and placed together with steel spheres in a rotating drum. When the drum rotates for a specific number of cycles, the aggregates damage when in contact with steel balls or other aggregate particles. Finally, the mass of crushed aggregates to smaller sizes was calculated and expressed as a percentage of the total mass of aggregates. Higher abrasion loss values indicate brittle aggregates.

Figure 3.9: Los Angeles abrasion machine

3.5.3 Flakiness and Elongation Index Tests

Aggregate particle shape (flat or elongated) is one of the most vital factors affecting the degradation of aggregates, especially in SMA mixtures. The flat or elongated aggregate particles refer to particles where the ratio of their width to their thickness or length is bigger than the specified value. However, in the SMA mixtures, both ratios were considered as there is a tendency to break during the construction process as a result of pressure exerted by heavy and frequent traffic loadings. The flakiness test for aggregate concerns the flat property of an aggregate compared with aggregate length. Aggregate particles are called flaky when the thickness is not greater than 0.6 of its mean size. The Flaky Index of an aggregate is calculated by the weight of flaky aggregate which passes a special opening as a percentage of the total aggregate tested. Flaky aggregates do not have sufficient strength and their strength is less than cubical aggregates and will not perform well as the mixture with cubical aggregates. The elongation of aggregates focuses on the length of the aggregates. The aggregate becomes elongated when the longest dimension (length) of the aggregates is 1.8 times greater than the mean dimension. Flakiness and elongation tests were performed according to the BS 812 Part 3 standard procedure with elongation and flakiness employ elongation and flakiness gauge, respectively as shown in Figure 3.10.

Figure 3.10: Elongation and flakiness gauges

3.5.4 X-ray Fluorescence

X-ray fluorescence (XRF) spectrometer (Axios-max) as shown in Figure 3.11 was used to measure the chemical oxide composition of POC and POFA.

Figure 3.11: Axios XRF spectrometer

This test was performed in the Geology Laboratory, Department of Geology, Faculty of Science, University of Malaya. To prepare the sample for testing, the POC and POFA materials were ground manually to obtain fine powder and sieved through a sieve of mesh size 0.075 mm (#200) as shown in Figure 3.12, the more finely ground the sample the more likely it is to be homogeneous and have limited void spaces providing for a better analysis. For each material (POC and POFA), a sample of 3.0 grams was mixed with 0.3 grams of cellulose wax as a binder. Then, the mixture (POC or POFA with cellulose wax) was filled into a pressing die and pressed to obtain a circular pellet sample. Finally, the sample was removed from the mould and kept in the laboratory for testing using XRF spectrometer. This method of sample preparation provides better analytical results than loose powders because the grinding and compression creates a more homogeneous representation of the material.

Figure 3.12: Material grinding using a mortar and a pestle

3.6 Marshall Mix Design

It is known that the key purpose of a mix design is to achieve an optimum binder content to produce a mixture with high resistance to deformation and cracking. When compared to other mix designs, the Marshall mix design method is the most common method used as it is lightweight, portable and requires inexpensive equipment.

Marshall mix design process consists of three simple steps: aggregate selection, asphalt binder selection, and optimum asphalt binder content determination. The Marshall compactor as shown in Figure 3.13 is capable of producing cylindrical specimens with a diameter of 102 mm and a height of 64 mm. The compaction pressure was specified at 457.2 mm with a free-fall drop distance of a hammer assembly with 4536 grams of sliding weight. The number of blows is highly dependent on traffic loading and can be adjusted to 35, 50 or 75 blows for each side.

Figure 3.13: Marshall compactor

In this study, 50 compaction blows were selected according to the Malaysian specification of the Public Works Department for road works (Jabatan Kerja Raya (JKR) Malaysia, 2008). Compaction blows of 75 were not used as it did not result in higher intensity than 50 blows. Besides, SMA mixtures were reported to be compacted easily to the desired density compared to traditional mixtures. (Al-Hadidy & Yi-qiu, 2009). Previous researchers have suggested application of 50 blows per side of each mixture instead of 75 blows in SMA mixture (Ahmadinia et al., 2012; Behnood & Ameri, 2012; Malarvizhi et al., 2012; Hainin et al., 2013; Mashaan et al., 2013; Rongali et al., 2013; Pourtahmasb & Karim, 2014b; Chen & Wei, 2016; Devulapalli et al., 2019, 2020; Huang et al., 2020).

3.6.1 Sample Preparation

Approximately 1100 grams of sample was weighed according to the combination of aggregates followed by heating to a temperature between 160 and 170 ºC for 2 hours. For the mixtures with replaced POC, the nature fine aggregates were replaced with POC at the desired percentage for each sieve size to maintain the gradation of aggregate. At the same time, asphalt binder was heated in another oven at a mixing temperature of 150-155 °C for 1 hour. All the ingredients were mixed using a dry process method. Required asphalt binder content (5.0%, 5.5%, 6.0%, 6.5% and 7.0%) by weight of mix was added and mixed aggregates were coated completely by the asphalt binder. The blended mixture was placed into a Marshall mould and moved to the Marshall compactor. Then, each sample was subjected to 50 blows on each side at a compaction temperature of 140°C-145°C according to the specifications of the Malaysian Public Works Department for road works (Jabatan Kerja Raya (JKR) Malaysia, 2008). Finally, the compacted samples were left to cool for 24 hours at a normal temperature and then were removed from the mould using a jack and kept in the laboratory until further testing.

3.6.2 Marshall Stability, Flow and Quotient

The Marshall stability of asphalt mixture is defined as the maximum load value used during the test before the compacted sample deforms at a temperature of 60 °C with a loading rate of 5.08 centimetres per minute (Mashaan et al., 2013). The Marshall stability was determined according to ASTM D6927 standard procedure (ASTM, 2015b). Meanwhile, Marshall flow value refers to plastic flow at the failure point of mix expressed in units of 0.25 mm and measured by sample vertical deformation in the direction of the applied load. The Marshall flow indicates the plasticity and flexibility properties of mixtures. According to the JKR specifications for SMA, the Marshall flow desired range is from 2 to 4 mm (Jabatan Kerja Raya (JKR) Malaysia, 2008). The minimum Marshall flow value indicates brittleness and strength, while the maximum Marshall flow value indicates the plasticity and maximum asphalt binder content of mixture (Akbulut & Gürer, 2007). Marshall quotient refers to the ratio between the Marshall stability value and the Marshall flow value which indicates asphalt resistance to rutting or permanent deformation (Gautam et al., 2018). Figure 3.14 shows the Marshall machine used to determine the stability and flow of asphalt mixture.

Figure 3.14: Marshall machine setup

3.6.3 Volumetric Properties

The volumes of asphalt binder and aggregates are affected by volumetric properties of asphalt mixture which in turn influence the pavement performance and durability (Jenks et al., 2011). In this study, volumetric properties such as density, air voids, and voids filled with asphalt binder were calculated. The density was determined according to ASTM D2726 standard procedure (ASTM, 2009), and the air voids and voids filled with asphalt binder were determined according to ASTM D3203 standard procedure (ASTM, 2011b).

3.6.4 Optimum Binder Content Selection

To optimise the asphalt mixtures, the Marshall standard procedure (ASTM D6926 (2010)) was used. Five percentages (5.0%, 5.5%, 6.0%, 6.5%, and 7.0%) of asphalt binder by the weight of mix were used and for each percentage, three duplicate samples were prepared using the Marshall compactor, each sample was subjected to 50 blows on each side instead of 75 blows as is the case in the dense-graded asphalt mixture. In dense-graded asphalt mixture, the selection of optimum binder content (OBC) depends on the stability, flow, specific gravity, and voids of the mixture. However, in SMA mixtures, the number of air voids were considered to be the main factor that affects the selection of optimum binder content (National Asphalt Pavement Association, 2002; Asi, 2006; Panda et al., 2013; Choudhary et al., 2018). Moreover, asphalt binder percentage that produces approximately 4% of air voids can cause lower asphalt binder bleeding with better rutting resistance (National Asphalt Pavement Association, 2002). In this study, the optimum binder contents for all SMA mixtures were selected to produce 4% of air voids and also following the mix design criteria specified by JKR (Jabatan Kerja Raya Malaysia (JKR), 2008). Statistical analysis was performed using SPSS software at a level of 5% significance.

3.6.5 Drain Down Test

Drain down condition refers to the portion of mixture (fine particles and asphalt binder) which separates and flows downward through the mixture which is more significant for SMA mixtures than the conventional (dense-graded) mixtures. This test was carried out following the ASTM D6390 standard procedure (ASTM, 2017). This test simulates the conditions of the mixture when it is produced, stored, transported, and placed on the roadway. The test is carried out on loose mixtures at the optimum binder content to ensure that the engineering properties of the binder drain down of the SMA mixture is within the required or acceptable level. The main procedure of the test is to place loose SMA mixture in a wire basket which was fabricated using a standard 6 mm sieve. The basket and the loose SMA mixture were then placed in an oven for one hour at 170 °C. The mass of fine particles and asphalt binder that drained from the mixture was measured and recorded as a percentage of the initial sample weight. The maximum drain down of SMA mixture is 0.3% by weight of the mixture (Jabatan Kerja Raya Malaysia (JKR), 2008).

3.7 Asphalt Mix Performance Tests

To evaluate the practical and mechanical properties of POC as a fine aggregate substitute in SMA20 mixture, the samples were prepared according to Marshall Mix design method. All the mixtures were tested for different performance tests. The mechanical performance tests carried out include resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability, and indirect tensile fatigue. The tests are described in detail in the following sub-sections.

3.7.1 Resilient Modulus Test

The resilient modulus test is the most appropriate and common test to measure the stiffness modulus of asphalt mixtures. It was carried out using a universal material testing apparatus (UMATTA) following ASTM D7369 standard procedure (ASTM, 2011a). Sample of an average of 101.7 mm in diameter and 65±1 mm in thickness was prepared for resilient modulus test. The test was carried out at three different temperatures (5 °C, 25 °C, 35 °C, and 40 °C). The samples were left in the chamber at the desired temperature for a minimum of 3 hours before the test was performed. In this test, all samples were tested at three different points of vertical loading and the average values were calculated and recorded. According to Zhang, Drescher, and Newcomb (1997), the resilient modulus can be calculated based on the following equation:

$$\text{Resilient Modulus (MPa)} = \frac{P}{\delta_H \times T}(I_1 - I_2 \times \nu) \qquad \text{………. (3.2)}$$

$$\nu = \frac{I_4 - I_1 \times \left(\frac{\delta_V}{\delta_H}\right)}{I_3 - I_2 \times \left(\frac{\delta_V}{\delta_H}\right)} \qquad \text{………. (3.3)}$$

where P is the load (N); v is the Poisson's ratio; T is the average thickness of the sample (mm); δ_H is the total recoverable deformation on the horizontal axis (mm) and δ_V is the total recoverable deformation on the vertical axis (mm) and "I_1, I_2, I_3 and I_4" are the constants which vary according to the gauge length positions.

A according to ASTM (2011a), in case of the gauge length as a fraction of specimen diameter = 1, I_1 = 0.27 and I_2 = -1.00. So, the stiffness modulus or resilient modulus of the samples can be calculated based on the following equation:

$$\text{Resilient Modulus (MPa)} = \frac{P(\nu + 0.27)}{\delta_H \times T} \qquad \text{………. (3.4)}$$

3.7.2 Wheel Tracking Test

Rutting is one of the common distresses that can happen on asphalt pavement, especially during hot climates. Various tests were conducted as reported in previous studies to assess rutting performance of pavement, including wheel tracking, Marshall, indirect tensile, and creep tests (Tayfur et al., 2007; Moghaddam et al., 2011). The wheel tracking test is the most common test employed as it offers better simulation outcomes on the field (Lu & Redelius, 2007). Wheel tracking test is used to determine the failure susceptibility of asphalt mixture due to weak aggregate structure, insufficient binder coating, and weak adhesion between binder and aggregates. In this study, this test was performed according to BS 598-110 standard procedure (British Standards Institute, 1998). The wheel tracking machine consisted of a steel wheel with a diameter of 200 mm and a width of 50 mm which moved in a constant wave at 42 passes or 21 cycles per min for 45 min. The 520±5 N load was applied to the centre line at the top surface of the asphalt mixture sample. Finally, the total rut depth value for each sample was recorded for every 105 cycles (5 minutes). The testing was stopped after 45 minutes or until the maximum rut depth reached 15.0 mm, or whichever occurred first.

In this study, both temperatures (60 °C and 45 °C) were selected. For each SMA20 mixture with POC replacement, two samples with the dimension of 200 mm × 300 mm with 50 mm thickness were prepared using a slab compactor. Table 3.4 presents the moderate to heavily, and very heavily stressed sites test requirements following BS 598-110 standard procedure. Figure 3.15 illustrates the slabs prepared using SMA20 mixtures with POC replacement.

Table 3.4: Wheel tracking performance requirement based on BS 598-110 standard

Parameter	Moderate to heavily stressed sites	Very heavily stressed sites
Temperature, °C	45	60
Maximum rut rate, mm/hr	2	5
Maximum rut depth, mm	4	7

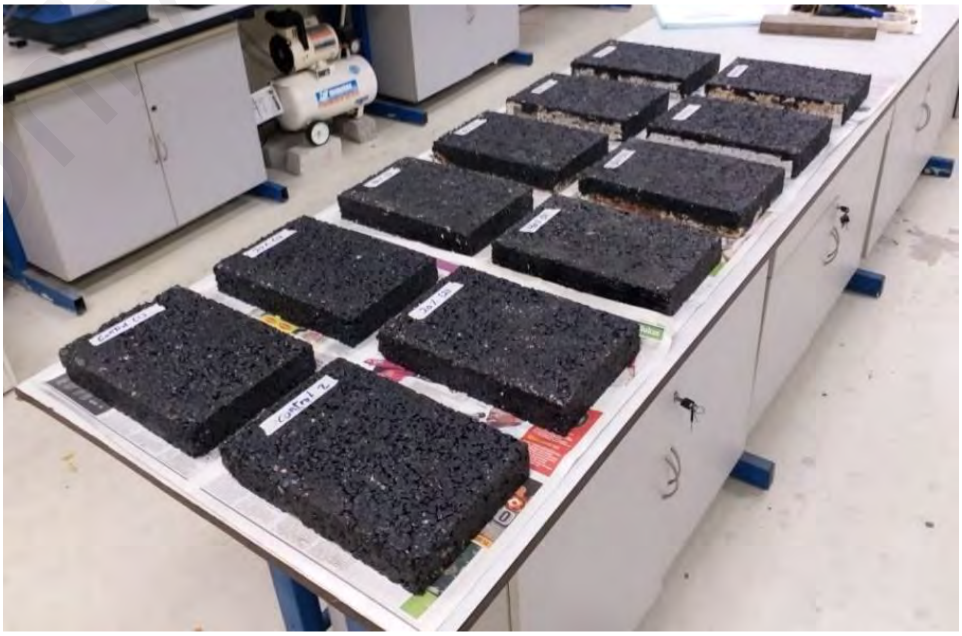


Figure 3.15: Prepared slab samples

3.7.3 Dynamic Creep Test

Dynamic creep test was performed to determine the rutting characteristics of asphalt mixtures by applying a dynamic load for thousands of cycles on a cylindrical Marshall specimen. Figure 3.16 displays the cumulative permanent deformation of asphalt mixture that was recorded as a function of the number of load cycles.

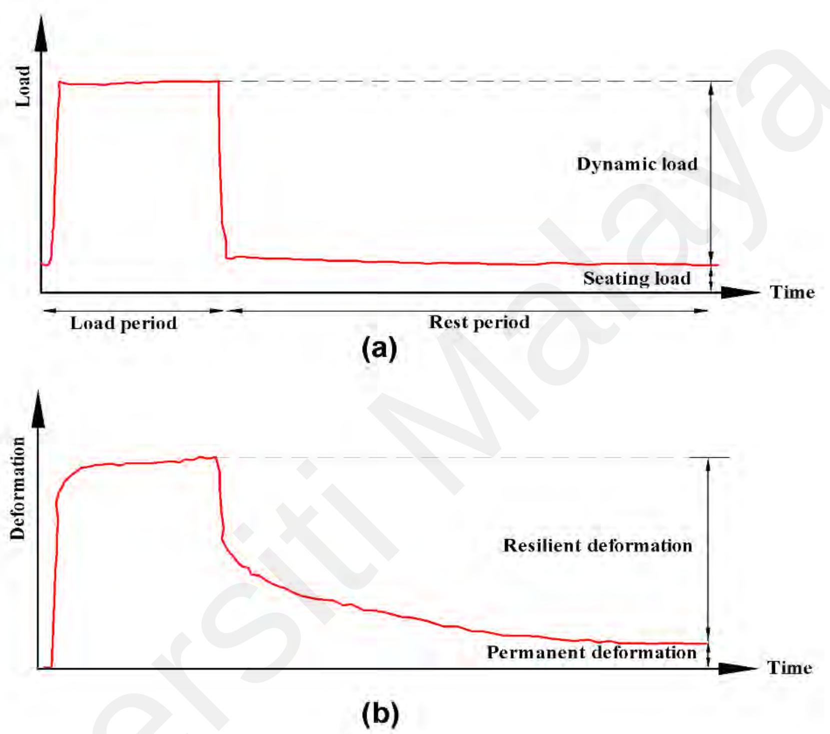


Figure 3.16: One cycle of load: (a) rectangular typical load, and (b) typical deformation

In this study, a creep test was carried out according to the Australian Standard (AS 2891.12.1) (Australian Standard, 1995) using UMATTA with stress control mode to eliminate the variance in sample cross-section. The samples with a dimension of 101.6 mm x 65±1 mm were cored from the top to the bottom using a diamond saw cutter to achieve a thickness of 50 ±1 mm. Each side was covered with a thin layer of grease followed by coating with graphite flakes to obtain a smooth surface. The samples were placed ins a machine chamber for two hours at 40 ºC as shown in Figure 3.17 before testing.

The samples were pre-loaded with static stress of 20 kPa for a minute and then were subjected to dynamic stress of 200 kPa for an hour. For each cycle, the load was applied for 0.5 seconds with a rest period of 1.5 seconds. The failure criterion of this test is either 1800 cycles or 100,000 micro-strains, whichever occurs first. To ensure precise results, a triplicate analysis was carried out for each asphalt mixture design.

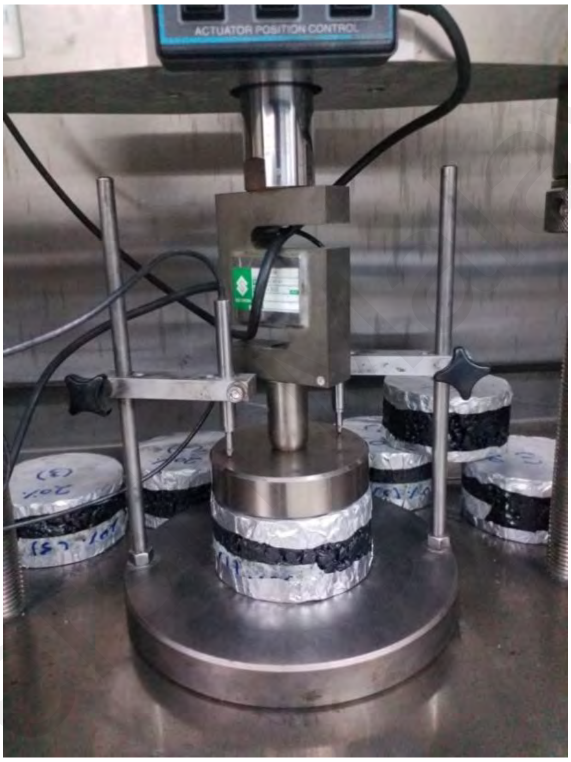


Figure 3.17: Setup for dynamic creep test

3.7.4 Indirect Tensile Strength Test

As stated in AASHTO-T283 (AASHTO, 2007), the purpose of the indirect tensile strength test is to determine the tensile strength and diametral direction of compacted hot mix asphalt. This test is commonly performed to determine the sensitivity of the mixture to moisture damage through tensile strength ratio (TSR) for dry and wet conditioned specimens. For each mixture, six specimens with a diameter of 101.6 mm and a depth of 65±1 mm were prepared and compacted to achieve 7.0% of air voids with three dry (unconditioned) and three wet (conditioned) samples. The wet condition of samples was achieved by submerging the samples in a water bath set at 60 ± 1 °C for 24 ± 1 h, followed by drying at room temperature (25 ± 0.5 °C) for 2h ± 10 minutes after removing the samples from water. After that, the load was applied in a diametral direction at a constant deformation rate of 50 mm/min as shown in Figure 3.18.

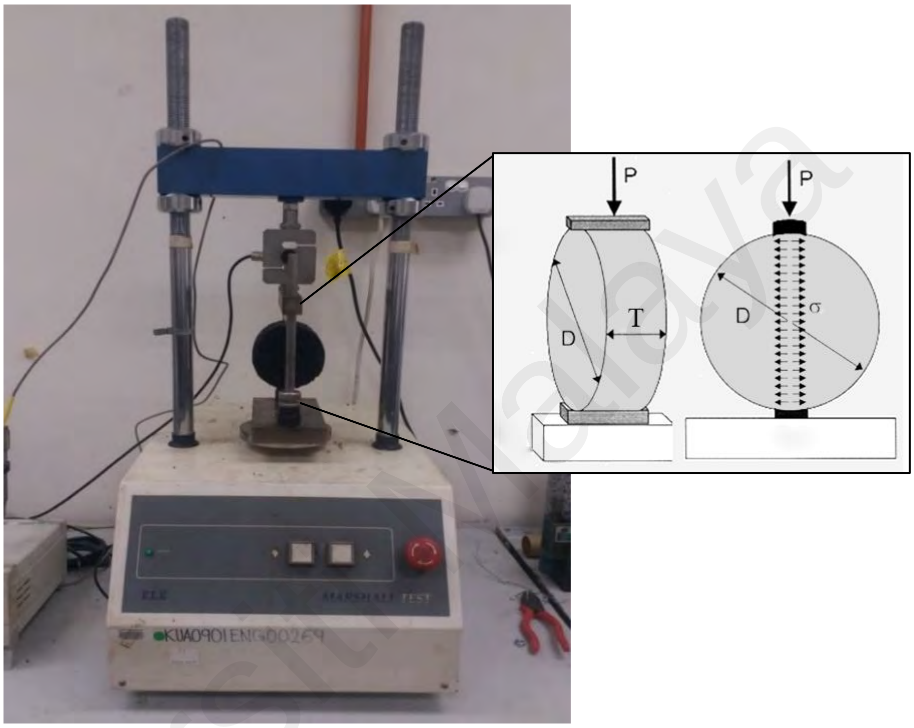


Figure 3.18: Machine setup for indirect tensile strength measurement

Hertz (1895) created mathematical (Equations 3.5-3.7) formulas to characterise the stress-states of elastic discs and spheres under diametral compression under point loading situation.

$$S_{t_x} = \frac{-2P}{\pi T}\left\{\frac{x^2(R-y)}{\beta_1^4} + \frac{x^2(R+y)}{\beta_2^4} - \frac{1}{2R}\right\} \quad \text{……………………… (3.5)}$$

$$S_{t_y} = \frac{-2P}{\pi T}\left\{\frac{(R-y)^3}{\beta_1^4} + \frac{(R+y)^3}{\beta_2^4} - \frac{1}{2R}\right\} \quad \text{……………………… (3.6)}$$

$$S_{t_{xy}} = \frac{2P}{\pi T}\left\{\frac{x(R-y)^2}{\beta_1^4} + \frac{x(R+y)^2}{\beta_2^4}\right\} \quad \text{……………………… (3.7)}$$

where $\beta_1^2 = (R-y)^2 + x^2, \beta_2^2 = (R+y)^2 + x^2$, S_t denotes tensile strength, P signifies maximum load, T refers to sample thickness, and R indicates sample radius.

The maximum principal stress occurs in the centre of the sample, and is tensile along the x-direction which responsible for the failure of the sample as shown in Figure 3.18. So, the tensile strength, St, is calculated by substituting values of x and y with 0 as follows:

$$S_t = \frac{2P}{\pi \times T \times D} \qquad \text{………………………. (3.8)}$$

where D indicates sample diameter

Finally, the indirect tensile strength dry value ($S_{t\text{-}dry}$) was divided with the indirect tensile strength wet value ($S_{t\text{-}wet}$) to determine the TSR value as expressed in Equation 3.9.

$$TSR = \frac{S_{t\text{ - Dry}}}{S_{t\text{ - Wet}}} \times 100 \qquad \text{……………………….. (3.9)}$$

A higher TSR value indicates better resistance of asphalt mix against moisture damage. As for SMA mixtures, the TSR must exceed 80% as recommended by AASHTO T 283 (AASHTO, 2007) and Malaysian Public Works Department for Road Works (Jabatan Kerja Raya (JKR) Malaysia, 2008).

3.7.5 Cantabro Durability Test

The Cantabro test was conducted to assess the resistance against abrasion loss or cohesion of asphalt mixture which was performed using Los Angeles abrasion by excluding steel balls in adherence to the European Standard (EN 12697-17) (British Standards Institution, 2017). A compacted sample of asphalt mixture with a diameter of 101.6 mm and a height of 65±1 mm was weighed and placed in a machine drum with a rotating speed of 30-33 rpm rotating speed. All the samples were conditioned at 25 °C before testing. Before continuing with a new test, debris on machine drum from previous tests were discarded. For every 100 revolutions, samples were brushed off and weighed. A similar procedure was conducted until 500 revolutions. The Cantabro Loss for each interval was calculated based on the following equation:

$$CL = \frac{A - B}{A} \times 100 \qquad \text{………………………. (3.10)}$$

where CL is Cantabro loss (%), while A and B denote initial and final sample weight (grams), respectively.

Figure 3.19 portrays images of untested and tested samples after every 100 revolutions.

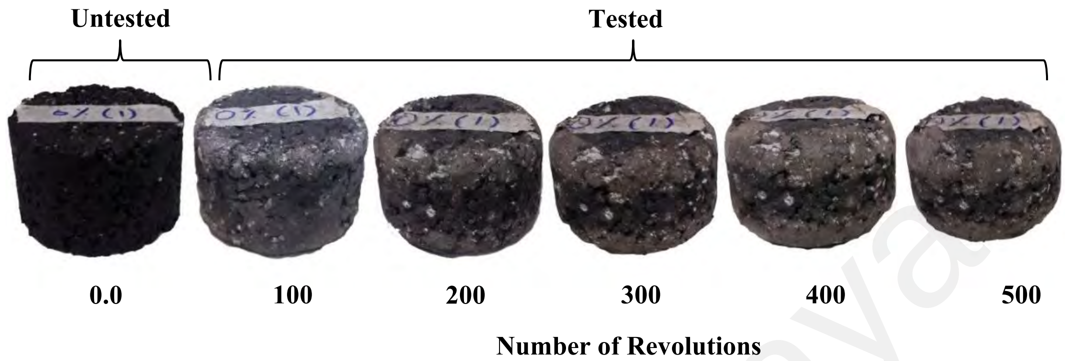


Figure 3.19: Visually changing shape and structure of mixture after mass loss

Based on previous findings, Cantabro test was reported to be sensitive to change in binder and aggregate properties of asphalt mixtures. Besides, several authors suggested that Cantabro test can be a tool to aid in the selection of material combinations concerning the durability of mixtures, especially mixtures with recycled materials. Cantabro test is an easy mixture test which can evaluate the durability of all mixes including SMA (Dong & Tan, 2011; Fauzi et al., 2020; Ferreira da Costa et al., 2020; Jasni et al., 2020) and dense-graded asphalt (Doyle & Howard, 2011; Cox, Smith, Howard, & James, 2017; Vila-Cortavitarte, Lastra-González, Calzada-Pérez, & Indacoechea-Vega, 2018).

3.7.6 Indirect Tensile Fatigue Tests

Asphalt mixture fatigue performance is linked to the lifespan of a roadway. Better fatigue performance of asphalt mixture indicates the longer lifespan of the pavement. The fatigue life of asphalt mixture depends on mixture properties such as grade and amount of asphalt binder, aggregate gradation, and a mixture of air voids (Moghaddam et al., 2012). The fatigue performance of asphalt mixture can be characterised by four-point bending fatigue test or indirect tensile fatigue test (Moghaddam et al., 2011). Indirect tensile fatigue test is an effective method that has been used in many studies to investigate the fatigue performance of asphalt mixture (Moghaddam et al., 2012; Mashaan et al., 2014; Modarres, Rahmanzadeh, & Ayar, 2015; Arabani & Pedram, 2016). This test can be carried out either in controlled stress or controlled strain mode. In the controlled stress mode, applied stress is kept constant while the strain values can be changed. In contrast, for controlled strain mode, the strain value is kept constant and the stress values can be changed. In this study, indirect tensile fatigue test was performed using a UMATTA machine in controlled stress mode according to EN 12697-24 standard procedure (British Standards Institution, 2018). The loading shape used was a haversine signal loading force of 2600 N with a loading time of 0.1s and a rest period of 0.4s. The test was carried out at 25 °C and the vertical deformation of the sample was recorded during the test. Fatigue life is defined as the number of load cycles reached when a sample cracks, or when a permanent vertical deformation reached the maximum value of 9 mm (Moghaddam et al., 2012).

Figure 3.20 shows the typical fatigue curve with three different deformation zones. The first part consists of initial load cycles and the fatigue curve has a power trend which could be explained by the occurrence of plastic deformation with a higher increment rate. For the second part, the rate of deformation gets stabilised and the fatigue curve has a linear trend. Micro-cracks formation in the second stage can progress to the next stage. For the third part, the spread and expansion of the cracks eventually resulted in a complete fracture of the sample. Two definitions of fracture life derived from indirect tensile fatigue tests are reflected in Figure 3.20. For the first definition, fracture life is equal to the total number of cycles that leads to a complete fracture of the sample. Meanwhile, for the second definition, fracture life corresponds to the intersection point of the second and third parts slopes (Modarres & Hamedi, 2014a).

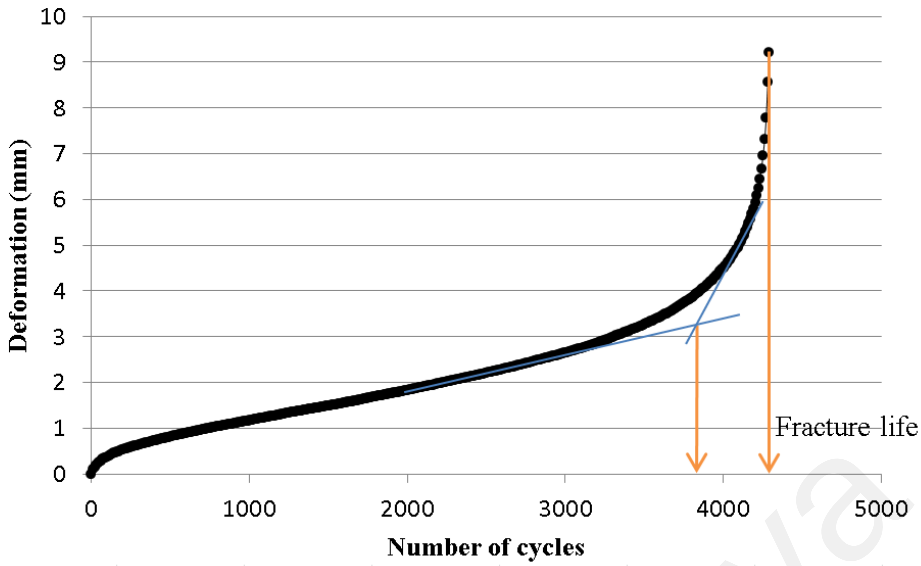


Figure 3.20: Fatigue curve and fracture life

3.8 Statistical Analysis

The current study aimed to evaluate the effect of POC as fine aggregates on the mechanical properties of asphalt pavement mixture. For this purpose, statistical analyses such as analysis of variance (ANOVA), regression and correlation were used to compare, relate, predict, and verify the findings using a statistical package for the social sciences (SPSS) software Version 24. The statistical tests applied are described in the following sections.

3.8.1 Analysis of Variance (ANOVA)

ANOVA is a statistical method employed for the evaluation of mean differences between three or more data sets. The analysis is a guide for researchers to determine the differences in a set of measurements or counts that are most likely expected from the variations of random chance, or, could be due to with a specific confidence level, certain factors influence the event. ANOVA is a useful and powerful tool to determine the statistical significance of differences (Field, 2009).

The significant level of data sets was determined by evaluating the F-ratio and comparing it to the F-critical value of the samples. If the F- ratio (F-statistic) is larger than F-critical, the variation between groups is considered statistically significant. The observed P-value is the probability of observing the F-ratio when the means of test results are equal. If the P-value is less than the desired level of significance (α), then the corresponding variant becomes significant (Mashaan, 2016a). The significance level (α) applied in this study is 0.05, representing a 5% probability of hypothesis represented by the model.

3.8.2 Pearson Correlation Analysis

Correlation analysis is also carried out in this study which can measure the strength of association (linear relationship) between two variables. The Pearson correlation coefficient (R) represents the slope of the best fit line of data relating to two variables. A correlation coefficient value should be in between −1 and +1. A coefficient value of +1 indicates a perfect positive correlation of two asphalt performance tests where, as one of them increases, the other increases by a corresponding amount. In contrast, a coefficient value of −1 indicates a perfect negative relationship, which means when the magnitude of one variable increases, the magnitude of another variable decreases. A coefficient value of 0 indicates no linear relationship (Field, 2009). In this study, Pearson correlation test was carried out between (i) rutting performance and (ii) fatigue performance with other mechanical properties, namely, resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performance.

The hypotheses for rutting performance and other mechanical properties performance are as follows:

- Null hypothesis, H_0: There is no correlation between rutting performance and other mechanical properties performance, namrly, resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performances.

- Alternative hypothesis, H_1: There is a correlation between rutting performance and other mechanical properties performance, namely, resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performances.

Tests of hypotheses for fatigue performance and other mechanical properties performance are as follows:

- Null hypothesis, H_0: There is no correlation between fatigue performance and other mechanical properties performance namely, resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performance.
- Alternative hypothesis, H_1: There is a correlation between fatigue performance and other mechanical properties performance, namely, resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performance.

3.8.3 Regression

Regression analysis is used to predict the value of a dependent variable based on a single predictor variable (simple regression) or several predictor variables (multiple regression). It explains the impact of changes of an independent variable on a dependent variable. This tool is incredibly useful where a specific association between data can be obtained (Field, 2009). The coefficient of determination (R^2) shows the degree to which the model describes the observed variance of the dependent variable. The R^2 is often ranged from 0 to 1, where the higher R^2 means a better match of the model.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

This chapter discusses the experimental results of this study on the effect of palm oil clinker (POC) utilisation as a fine aggregates substitute on SMA mixture properties. This chapter is divided into three main parts where the first part presents the physical properties of aggregates and asphalt binder used in the study. The second part discusses the Marshall mix design and optimum asphalt binder content required for different SMA mixtures. Meanwhile, the final part elaborates on the mechanical properties of SMA mixtures which includes the resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability and indirect tensile fatigue tests.

4.2 Physical Properties

4.2.1 Asphalt binder

The asphalt binder with a penetration grade of 80/100 was used in this study. Physical properties of asphalt binder such as penetration, softening point, rotational viscosity, specific gravity and DSR are as in Table 4-1.

Table 4-1: Properties of asphalt binder with a penetration grade of 80/100

Properties	Temp.	Unit	Value	Reference	Requirement
Penetration	25˚C	0.1 mm	87	ASTM-D0005	80-100
Softening Point	-	°C	46	ASTM-D0036	45-52
Ductility	25˚C	cm	+ 100	ASTM D0113	Min. 100
Rotational Viscosity	135˚C	Pa.s	0.312	ASTM-D4402	< 3.0
Rotational Viscosity	165˚C	Pa.s	0.100	ASTM-D4402	< 3.0
Specific Gravity	25˚C	-	1.020	ASTM-D0070	1.01-1.06
G*/ sinδ	58˚C	kPa	1.576	ASTM-D7175	> 1.0
G*/ sinδ	64˚C	kPa	0.714	ASTM-D7175	> 1.0

Penetration value of 87 was obtained which was in the range of the specified grade, 80-100. The softening point temperature was approximately 46 °C. The viscosities of asphalt binder at 135 ˚C and 165 ˚C were 0.31 and 0.10 Pa.s, respectively which were lower than the maximum viscosity limit, i.e., 3.0 Pa.s, according to AASHTO M320. This indicated that the use of asphalt binder can result in a homogenous mixture and good coating of aggregates.

According to SUPERPAVE™ specification, the permanent deformation (rutting) is governed by the parameter G*/sinδ which should be greater than or equal to 1.00 kPa for the original asphalt binder. From Table 4-1, it can be seen that the 80/100 asphalt binder satisfied the standard rutting parameter (G*/ sinδ) at 58˚C, but did not meet the specification at 64˚C. Therefore, based on performance grade (PG) classification, the 80/100 asphalt binder can be classified as PG58.

4.2.2 Aggregates

Aggregate quality is one of the key factors that affect the performance of asphalt mixtures. Aggregate properties such as strength and gradation significantly affect the performance of asphalt (Abo-Qudais & Al-Shweily, 2007). Therefore, the quality of aggregates should be evaluated using different physical properties tests. The granite aggregates tests results are presented in Table 4.2. From the results, it can be noted that the aggregates satisfied the typical requirements and indicated its suitability for application in the asphalt pavement. In the present study, X-ray fluorescence (XRF) analysis was carried out to determine the chemical components and physical properties of POC and POFA and the findings are presented in Table 4.3.

Table 4.2: Physical properties of granite aggregates

Parameter	Test Method	Value	Standard Requirement
Los Angeles abrasion (%)	ASTM C-131	19.8	< 30 %
Flakiness index (%)	BS 812: Part3	7.3	< 20%
Elongation index (%)	BS 812: Part3	13.4	< 20%
Impact value (%)	BS 812: Part3	10.3	< 15%
Specific gravity of coarse aggregates	ASTM C-127	2.61	-
Water absorption of coarse aggregates (%)	ASTM C-127	0.65	< 2%
Specific gravity of fine aggregates	ASTM C-128	2.59	-
Water absorption of fine aggregates (%)	ASTM C-128	1.19	< 2%

Table 4.3: Chemical components and physical properties of POC and POFA

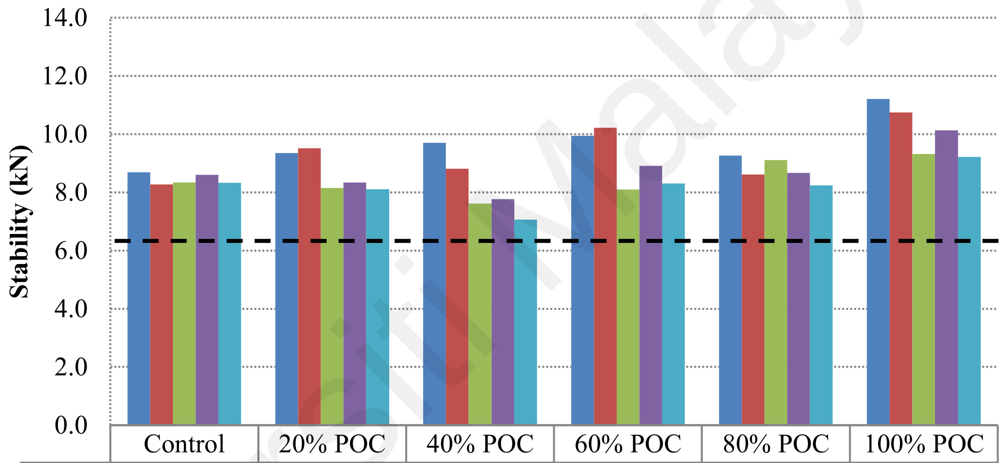
Properties	Unit	POC	POFA
Silicon dioxide (SiO_2)	%	65.4	68.3
Aluminium oxide (Al_2O_3)	%	1.947	2.862
Iron oxide (Fe_2O_3)	%	2.713	4.641
Calcium oxide (CaO)	%	5.744	4.906
Magnesium oxide (MgO)	%	6.402	4.718
Sodium oxide (Na_2O)	%	0.318	0.221
Potassium oxide (K_2O)	%	9.519	6.953
Sulfur trioxide (SO_3)	%	0.635	1.501
Phosphorus pentoxide (P_2O_5)	%	6.563	4.435
Titanium dioxide (TiO_2)	%	0.113	0.20
Manganese oxide (MnO)	%	0.165	0.096
Chloride (Chl)	%	0.113	0.912
Specific gravity	-	2.080	2.20
Colour	-	Light grey	Dark grey

4.3 Marshall Test

In this study, six groups of asphalt mixtures were prepared using different amount of POC (0%, 20%, 40%, 60%, 80%, and 100% of the fine aggregate weight) as the fine aggregate replacer. For each group, three duplicate samples with a constant content of POC and different content of asphalt binder (5.0%, 5.5%, 6.0%, 6.5%, and 7.0%) were prepared to determine the Marshall stability, Marshall flow, and volumetric properties. The Marshall mix design results obtained for all the different mixtures are presented in Tables A.1 to A.6 in appendices.

4.3.1 Marshall Stability

The average Marshall stability of three duplicate samples from each asphalt mixture is presented in Figure 4.1.



	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	8.69	9.34	9.70	9.94	9.26	11.21
5.5%	8.27	9.52	8.81	10.22	8.61	10.75
6.0%	8.34	8.16	7.62	8.10	9.11	9.31
6.5%	8.61	8.33	7.76	8.91	8.67	10.13
7.0%	8.33	8.11	7.06	8.30	8.24	9.22

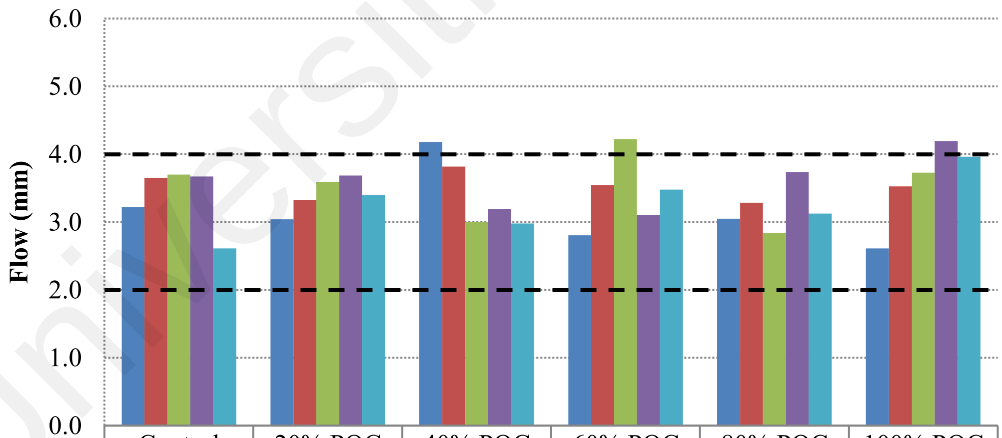
Figure 4.1: Marshall stability as influenced by different POC and asphalt binder contents

The results indicated that the addition of asphalt binder into the mixture resulted in a decrease in the stability value that could be ascribed to the reduced contact point between the aggregates within the asphalt mixture (Mashaan et al., 2013). Moreover, the stability values for all asphalt binder contents vary in tandem with the POC content. Based on the literature review, Marshall stability and flow tests are not suitable strength tests for SMA mix and the results can be used as a piece of information and should not be the sole reason to accept or reject an SMA design (National Asphalt Pavement Association, 2002; Panda et al., 2013). The SMA mix stability mainly relies on the stone-to-stone contact in the matrix which leads to the unconventional trend of the Marshall stability and flow performance. A similar trend was also observed in previous studies (Moghaddam & Karim, 2012; Moghaddam, 2013; Sargın et al., 2013; Mashaan, 2016b).

Figure 4.1 shows that the control mixture and aggregate replacement mixtures satisfied the recommended minimum stability value of 6.2 kN according to the JKR specifications for SMA20 (Jabatan Kerja Raya Malaysia (JKR), 2008). Therefore, POC can be an alternative waste material to replace fine aggregate in SMA20.

4.3.2 Marshall Flow and Quotient

The Marshall flow values and POC contents for different percentage of asphalt binder are illustrated in Figure 4.2.



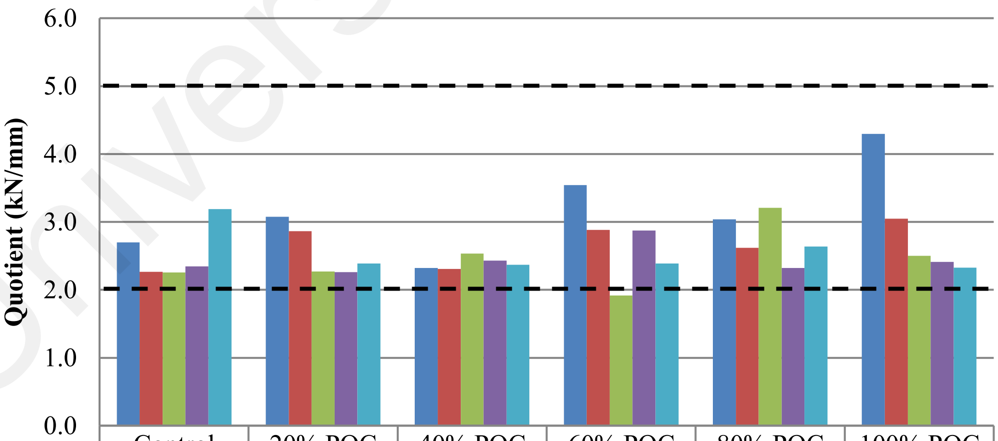
	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	3.22	3.04	4.18	2.81	3.05	2.61
5.5%	3.65	3.33	3.82	3.55	3.29	3.53
6.0%	3.70	3.59	3.00	4.22	2.84	3.73
6.5%	3.67	3.69	3.19	3.10	3.74	4.20
7.0%	2.61	3.40	2.98	3.48	3.12	3.96

Figure 4.2: Marshall flow for different POC and asphalt binder contents

The results showed that there was no clear trend observed on the influence of POC contents on the flow values. However, most of the asphalt mixtures' Marshall flow values were within the acceptable range (2 - 4mm) as recommended by the JKR for SMA20

(Jabatan Kerja Raya Malaysia (JKR), 2008). Nevertheless, several SMA mixes with POC replacement showed slightly higher values than the requirement. The mixtures were (1) SMA mix with 40% of POC and 5.0% of asphalt binder, (2) SMA mix with 60% of POC and 6.0% of asphalt binder, and (3) SMA mix with 100% of POC and 6.5% of asphalt binder with flow values of 4.18mm, 4.22mm and 4.2 mm, respectively.

Figure 4.3 shows the Marshall quotient results as affected by different POC and asphalt binder contents. According to Indian specification, the acceptable range of Marshall quotient is 2 to 5 kN/mm (MORTH, 2013). Most of the asphalt mixtures quotient values were within the acceptable range of 2 and 5 kN/mm, except for the mix with 60% of POC and 6.0% of asphalt binder where the value was slightly lower than the prescribed lower limit. Therefore, it can be concluded that asphalt mixtures with POC as fine aggregate substitute possess required strength and stiffness to resist permanent deformation under heavy traffic loads.

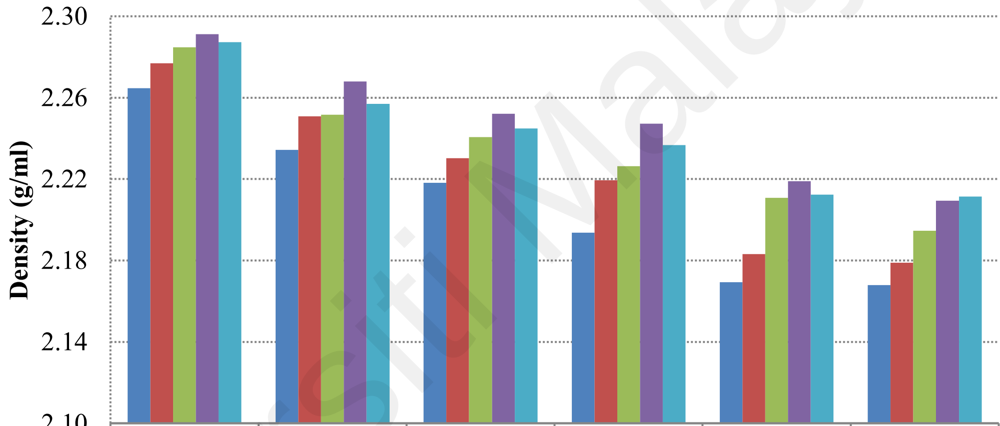


	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	2.70	3.07	2.32	3.54	3.04	4.30
5.5%	2.26	2.86	2.31	2.88	2.62	3.05
6.0%	2.25	2.27	2.54	1.92	3.21	2.50
6.5%	2.35	2.26	2.43	2.87	2.32	2.41
7.0%	3.19	2.39	2.37	2.39	2.64	2.33

Figure 4.3: Marshall quotient for different POC and asphalt binder contents

4.3.3 Mixture Density

The density values of all the mixtures as affected by different POC contents for different asphalt binder contents are graphically presented in Figure 4.4. The results showed that the POC as a fine aggregate replacer affected the density of the mixture. For any asphalt binder content, the density value of the compacted sample decreased with an increase in the POC aggregate replacement. Moreover, all the mixtures with the POC as an aggregate substitute possessed lower density compared to the control mix that could be due to the lower specific gravity of POC compared to natural aggregates.



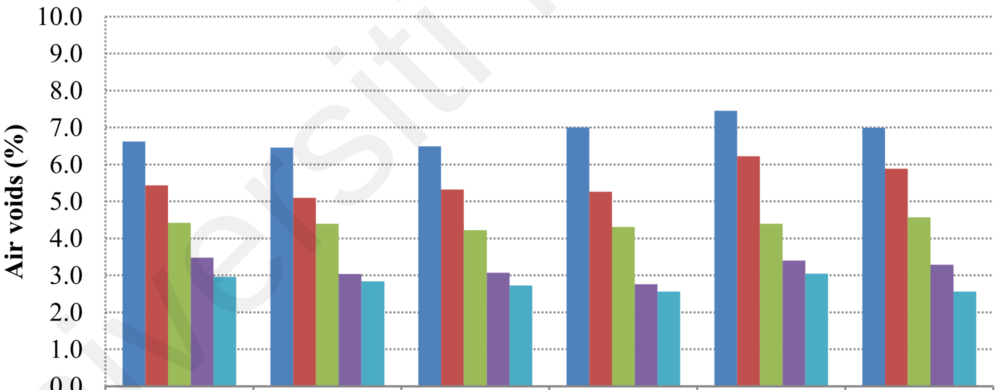
	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	2.265	2.234	2.218	2.194	2.169	2.168
5.5%	2.277	2.251	2.230	2.219	2.183	2.179
6.0%	2.285	2.252	2.241	2.226	2.211	2.195
6.5%	2.291	2.268	2.252	2.247	2.219	2.209
7.0%	2.287	2.257	2.245	2.237	2.212	2.211

Figure 4.4: Mixture density as affected by different POC and asphalt binder contents

Moreover, the trend of mixture density was almost similar for all the different treatment groups of POC with varying levels of asphalt binder. The density kept increasing until 6.5% of asphalt binder which reduced thereafter. The main reason for the density increment could be attributed filling of air voids between the aggregates by the asphalt binder. Meanwhile, the decrease in the density beyond the use of 6.5% of asphalt binder was due to saturation of air voids between the aggregates by the asphalt binders.

4.3.4 Air Voids and Void Filling by Binders

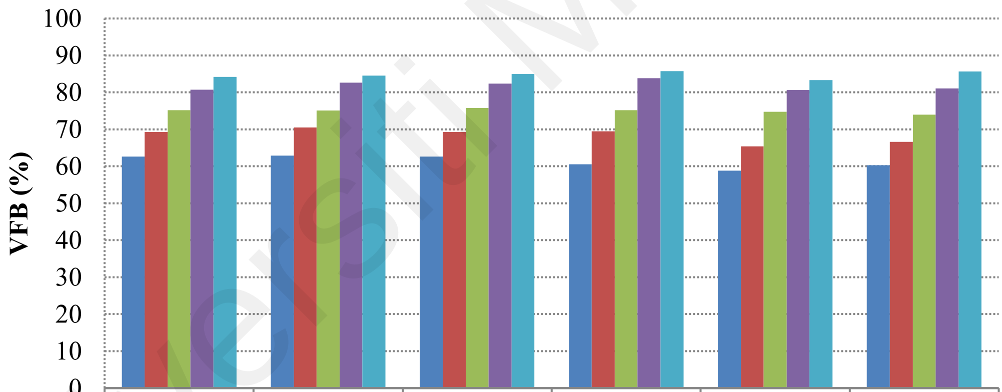
Amount of void in the mix is one of the important parameters for asphalt mix designs that can be used to determine the optimum content of asphalt binder required for SMA mixes (National Asphalt Pavement Association, 2002; Asi, 2006; Panda et al., 2013). The performance of the asphalt mixture is dependent on the number of air voids in the mix. According to National Asphalt Pavement Association (2002) and Jabatan Kerja Raya (JKR) Malaysia (2008) the acceptable limit of air voids is 3-5%, too many air voids in the mixture can lead to asphalt cracking due to low asphalt binder content leading to failure in coating the aggregates in the mixture. In contrast, a low amount of air voids in the mixture may result in more deformation and asphalt binder bleeding (Ahmadinia et al., 2011). The effect of POC and binder content on the number of air voids in the mixtures are presented in Figure 4.5.



	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	6.62	6.46	6.49	7.01	7.46	7.00
5.5%	5.44	5.10	5.32	5.27	6.23	5.89
6.0%	4.43	4.40	4.23	4.32	4.40	4.57
6.5%	3.48	3.04	3.07	2.76	3.40	3.29
7.0%	2.96	2.84	2.72	2.56	3.04	2.56

Figure 4.5: Effect of different POC and asphalt binder contents on the mixture air voids

From Figure 4.5, the trend of air voids data was almost similar despite different POC contents indicating replacement of aggregate with POC aggregate does not affect the air void values of mixtures. However, there was a significant effect on the mixtures' air voids detected for different asphalt binder contents, where the air void values decreased with the increase in asphalt binder content. A similar finding was reported in several previous studies (Asi, 2006; Akbulut & Gürer, 2007; Ahmadinia et al., 2011; Abdul-Rahman & Abdul-Wahab, 2013; Panda et al., 2013; Acosta Álvarez, Alonso Aenlle, Tenza-Abril, & Ivorra, 2020). The decrease in the air voids can be explained by the phenomenon of air voids filling by asphalt binders. Of all the mixtures with replaced POC, mixtures containing 6.5% and 7.0% binders were not acceptable as the number of air voids was lower than the desired value, i.e. 4.0%, which resulted in more deformation and asphalt binder bleeding. On the contrary, a higher amount of air voids resulted when a lower amount of asphalt binders (5.0% and 5.5%) were used which caused cracking of asphalt pavement. The results for the percentage of voids filled with asphalt (VFA) as affected by different POC and asphalt binder contents are presented in Figure 4.6.



	Control	20% POC	40% POC	60% POC	80% POC	100% POC
5.0%	62.64	62.93	62.64	60.55	58.79	60.32
5.5%	69.31	70.49	69.32	69.45	65.42	66.65
6.0%	75.22	75.07	75.84	75.22	74.74	73.95
6.5%	80.77	82.64	82.36	83.83	80.63	81.07
7.0%	84.18	84.53	84.98	85.73	83.32	85.63

Figure 4.6: VFB in mixtures with different binder and POC contents

It was found that for all asphalt binder contents evaluated, an increase in POC content caused the VFA to stabilise which was indicative that a change in the POC content did not affect the VFA values. However, there was a clear effect of asphalt binder content on VFA values. The VFA values increased when the asphalt binder contents increased which was consistent with that reported in several previous studies (Asi, 2006; Akbulut & Gürer, 2007; Abdul-Rahman & Abdul-Wahab, 2013).

4.3.5 Asphalt Content Optimisation

For each asphalt mixture group, five percentages of asphalt binder (5.0%, 5.5%, 6.0%, 6.5%, and 7.0% of the total weight of mix) were used to determine the optimum binder content (OBC). The OBCs for all the SMA mixtures were selected to produce 4% of air voids. The OBCs for the mixtures are shown in Figure 4.7. Besides, volumetric properties at OBC of asphalt mixtures such as density, voids in mineral aggregate (VMA) and voids filled with asphalt (VFA) are presented in Table 4.4.

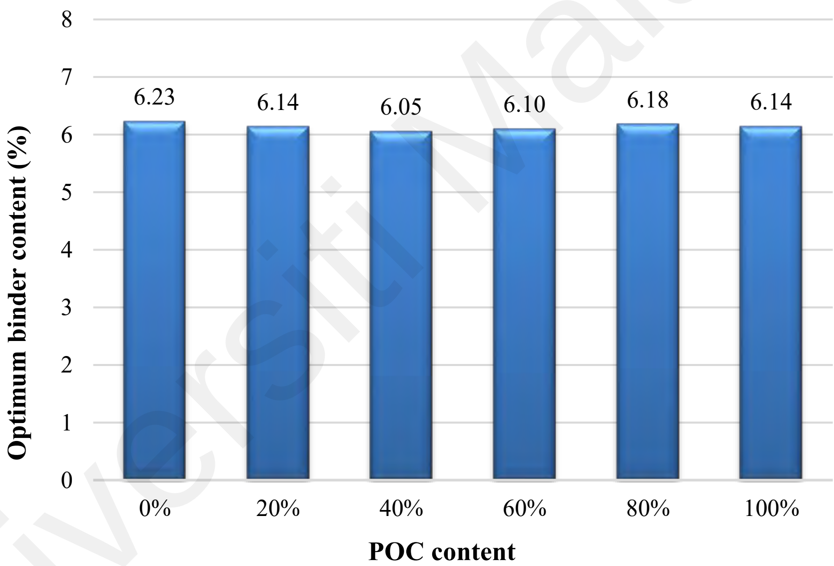


Figure 4.7: Optimum binder content (OBC) for different asphalt mixtures

Table 4.4: Properties of mixtures at optimum binder content

Mixture	OBC (%)	Density (g/ml)	VMA (%)	VFA (%)
Control	6.23	2.288	17.96	77.77
POC-20	6.14	2.256	17.60	77.19
POC-40	6.05	2.239	17.50	75.96
POC-60	6.10	2.231	17.35	76.94
POC-80	6.18	2.214	17.45	76.86
POC-100	6.14	2.199	17.45	75.94

OBC: *optimum binder content,* **VMA:** *voids in mineral aggregate,* **VFA:** *voids filled with asphalt*

Statistical analysis was carried out using analysis of variance (ANOVA) test to evaluate the effect of POC on the OBC. The hypothesis test was applied to determine the presence of significant difference of OBC between all the mixtures. The null hypothesis (H_0) specified was μ1=μ2=μ3=μ4=μ5=μ6 where μ1, μ2, μ3, μ4, μ5, and μ6 represent the mean value of OBC for different mixtures, and the alternative hypothesis (H_1) was at least one mean value of OBC for different mixtures induces a difference. The descriptive statistics results are shown in Table 4.5, and the ANOVA results are shown in Table 4.6. It can be seen that the differences in the OBC were not statistically significant since the p-value =0.578 which, was higher than the alpha value (0.05). Also, the F-ratio (0.787) value was less than 1 which indicated an insignificant difference. In SMA mixtures, the number of air voids were considered to be the main factor that affects the selection of optimum binder content. In this study, the optimum binder content was selected to produce 4% of air voids, and as sown in Figure 4.5 different POC contents do not affect the air void values of mixtures. For this reason, the use of POC in the mixtures did not affect the OBC, and in turn, there would be no additional cost resulting from the use of more asphalt binder.

Table 4.5: Descriptive statistics results for optimum binder content

Mixtures	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
0%	3	6.23	0.038	0.022	6.13	6.32	6.18	6.25
20%	3	6.14	0.070	0.041	5.96	6.31	6.07	6.21
40%	3	6.05	0.194	0.112	5.57	6.53	5.91	6.27
60%	3	6.10	0.046	0.026	5.98	6.21	6.06	6.15
80%	3	6.18	0.094	0.055	5.94	6.41	6.07	6.26
100%	3	6.14	0.175	0.101	5.71	6.58	5.98	6.33
Total	18	6.14	0.116	0.027	6.08	6.20	5.91	6.33

Table 4.6: ANOVA results for optimum binder content

	Sum of Squares	DF	Mean Square	F Ratio	P-value
Between Groups	0.056	5	0.011	0.787	0.578
Within Groups	0.172	12	0.014		
Total	0.228	17			

4.3.6 Drain Down Test Results

The susceptibility of SMA mixtures with different levels of POC replacement to drain down was evaluated. The results of drain down performance for control mix and SMA mixtures with the replacement of POC are presented in Figure 4.8 (See Table A. 7 in Appendices for more details).

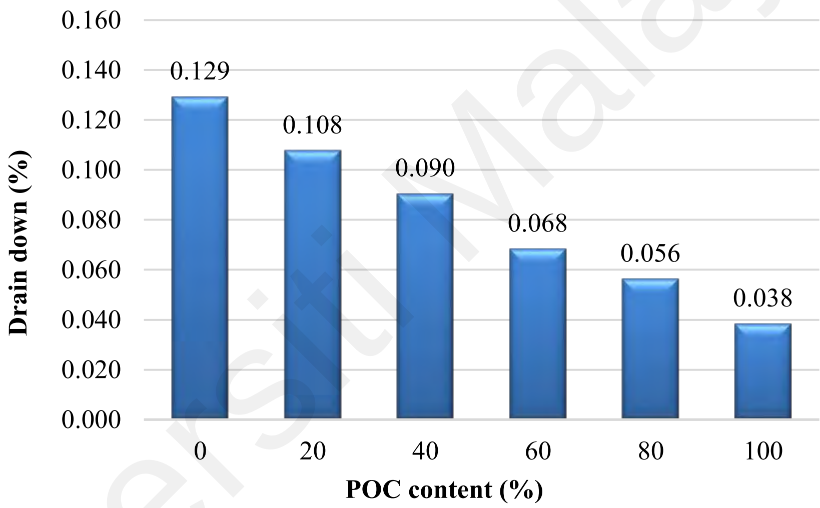


Figure 4.8: Binder drain down for different SMA mixtures.

It can be observed that all SMA mixtures with different levels of POC replacement exhibited lower drain down values as compared to maximum allowable limit, i.e., 0.3% by total weight of the mixture. The percentage of drain down decreased significantly with the increment in POC content. The SMA mixture with 100% of POC replacement showed the lowest drain down performance, i.e., 0.038% by total weight of the mixture. For all mixtures replaced with POC, the binder drain down (%) were lower compared to the control mixture (0% POC replacement) which indicated that POC functions as a stabilising additive in SMA mixtures. The lower values of drain down of SMA mixtures with the replacement of could be due to the existence of fibres that originated from oil palm waste. The fibres were reported to reinforce the SMA mixtures and assist in generating a three-dimensional network which could improve the mix adhesion and reduce the percentage of asphalt binder drain down (Slebi-Acevedo, Lastra-González, Pascual-Muñoz, & Castro-Fresno, 2019). This finding is also consistent with that reported by Panda et al. (2013), where the use of coconut fibre in SMA mixture was noted to prevent the usual draining of asphalt binder.

4.4 Mechanical Properties of Asphalt Mixtures

To evaluate the effectiveness of POC, SMA mixtures were prepared according to the Marshall mix design method. Mixtures with substituted POC and without POC (control) were analysed for different performance tests. The results are described in the following sub-sections.

4.4.1 Resilient Modulus of Asphalt Mixtures

The average value of resilient modulus of each SMA mixture with substituted POC and control SMA mixture are presented in Figure 4.9. Resilient modulus values increased with the increase in the POC replacement until reach to the maximum (at 40% of POC replacement) and then decrease. It is desired to have higher stiffness values at a higher temperature to improve rutting resistance, while low stiffness values are preferred at a lower temperature for thermal cracking resistance. The increment in stiffness values at a high temperature for SMA mixtures with the replacement of POC proposed a potential utilisation of POC as a fine aggregate in asphalt mixtures. The increment in the resilient modulus can be related to the existence of fibrous material in crushed POC. Fibres help the mixture to resist tensile strength which can prevent the expansion of cracks (Lavasani, Namin, & Fartash, 2015; Slebi-Acevedo et al., 2019), hence increasing the stiffness of the samples. However, the inclusion of a higher amount of fibre can cause reduced adherence between the mixture compositions which can lead to a reduction in stiffness.

Maximum resilient modulus was exhibited by the mixture that contains 40% of POC. Overall, all the SMA20 mixtures with POC replacement have higher resilient modulus as compared to the control mix. As displayed in Table 4.7, SMA20 mixtures containing 40, 60 and 80% of POC as the fine aggregate replacer possessed higher resilient modulus by 1.18, 1.16 and 1.15 times, respectively compared to the control mix. This resulted in increased rutting resistance of mixtures containing 40 - 80% of POC. A similar trend was observed at a temperature of 25 °C by previous studies where SMA mixture was prepared using recycled concrete as a fine aggregate replacer (Pourtahmasb & Karim, 2014a) and shredded waste plastic with a particle size of 2.36 mm (Ahmadinia et al., 2012; Moghaddam & Karim, 2012).

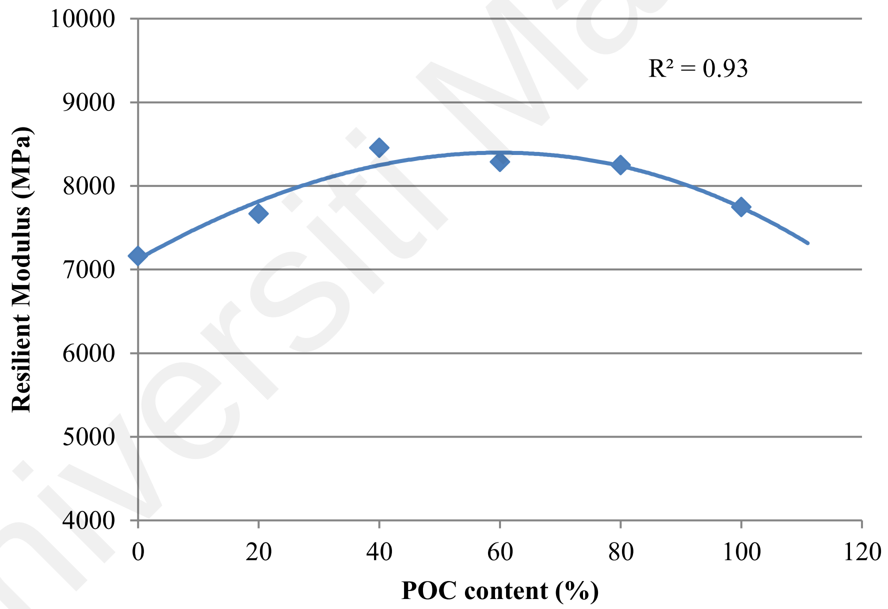


Figure 4.9: Resilient modulus of mixtures with different POC content

Table 4.7: Normalised resilient modulus of different POC mixtures

Mixtures	Resilient Modulus (MPa)	Normalised Resilient Modulus
Control mix	7163	1.00
POC-20	7665	1.07
POC-40	8454	1.18
POC-60	8285	1.16
POC-80	8247	1.15
POC-100	7749	1.08

Besides, the effect of temperature susceptibility on the resilient modulus exhibited a similar trend for all the different groups of POC as shown in Figure 4.10 (See Tables A.8 to A.13 in Appendices for more details). As the temperature increased, the resilient modulus of the control mixture and SMA mixtures with POC replacement decreased. This observation can be related to the decrement in the asphalt binder viscosity as a result of the increment in temperature that caused particle slippage in asphalt mixtures.

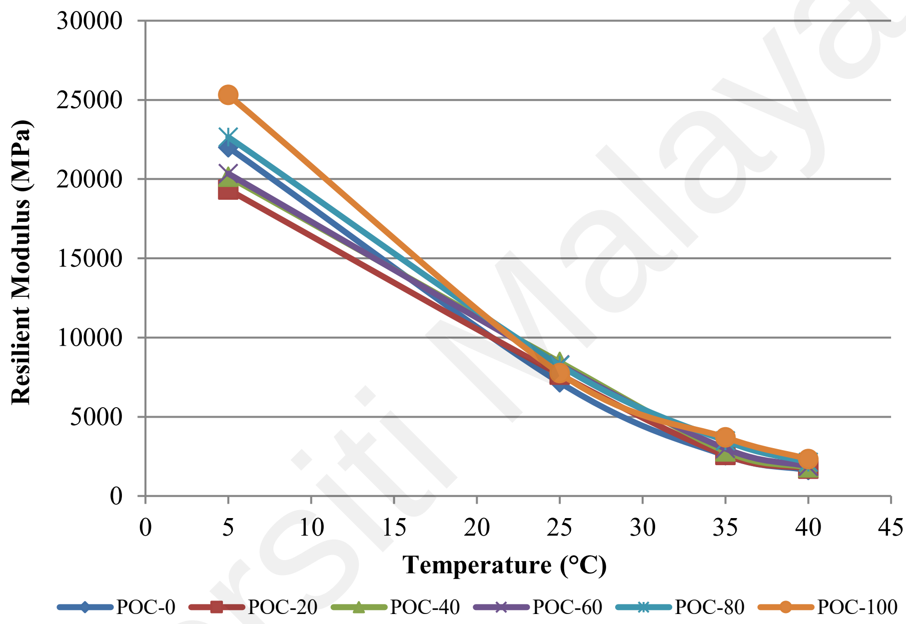


Figure 4.10: Effect of temperature on resilient modulus of POC mixtures

4.4.2 Wheel Tracking Test Results

For each mixture, rut depth values of slabs which were retrieved from the wheel tracking test for every 5 minutes are shown in Tables A.14 to A.19 in Appendices. The mean rut depth versus time for all SMA20 mixtures are presented in Figure 4.11 and Figure 4.12 at 45 °C and 60 °C, respectively.

The permanent deformation values for all SMA20 mixtures were lower than that of the control mix which signified higher rutting resistance of mixtures with substituted POC compared to the control mix. This could be attributed to the presence of fibrous material in crushed POC.

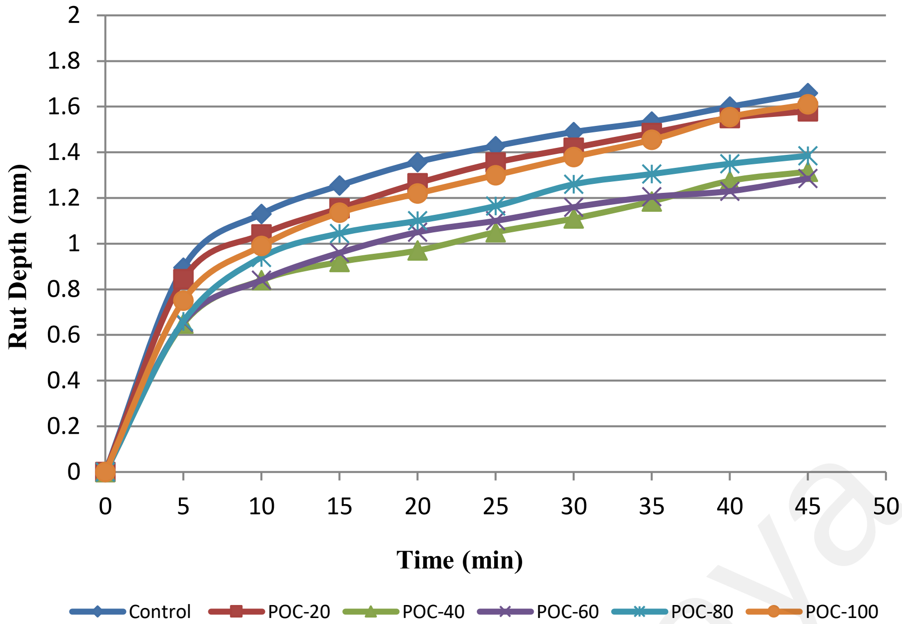


Figure 4.11: Rut depth versus time for different mixtures at 45ºC

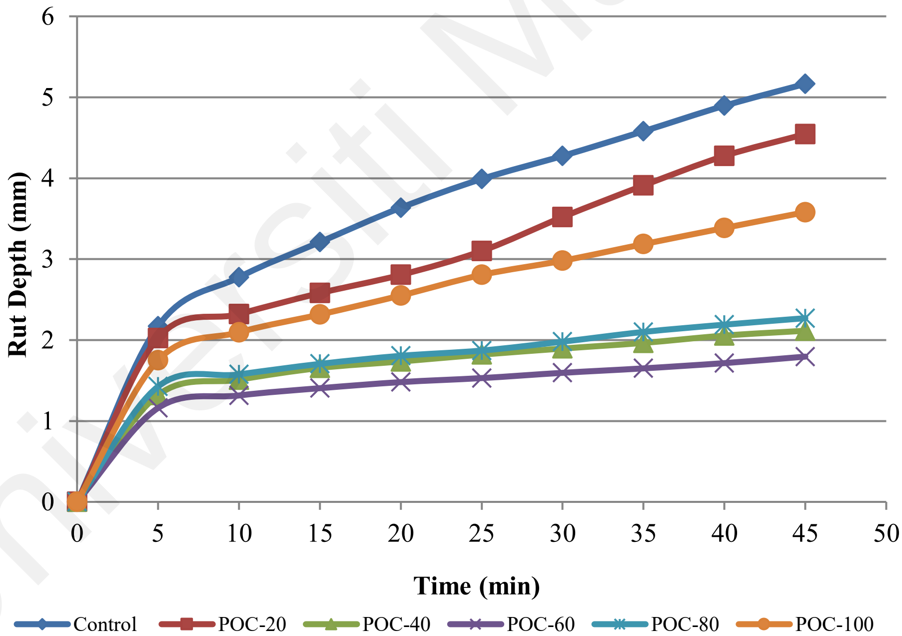


Figure 4.12: Rut depth versus time for different mixtures at 60ºC

The cumulative rut depth data after 45 minutes of testing are graphically presented in Figure 4.13 . The SMA20 mixtures with POC replacement led to the decrease in the rut depth until the minimum value was achieved followed by a subsequent increase in rut depth values. Comparison of normalised ruth depth values as displayed in Table 4.8 showed that asphalt mixture replaced with 60% of POC emerged as the best mixture, which was followed by 40%. The rut depth values of mixtures contain 40% and 60% POC at 45 °C decreased by 0.79 and 0.77 times, respectively when compared to the control mix. Meanwhile, the rut depth values at 60 °C decreased by 0.41 and 0.35 time for mixtures contain 40% and 60% POC, respectively. POC contents were noted to exert a minor effect at 45 °C but show significant variations at 60 °C. This observation was due to the reduction in asphalt binder viscosity as a result of the increment in temperature that led to particle slippage in asphalt mixtures. Based on the British Standard for the determination of wheel-tracking rate and depth (BS 598-110) (British Standards Institute, 1998), 4.0 mm and 7.0 mm are the maximum values of rut depth at 45 °C and 60 °C, respectively (Table 3.4). As depicted in Table 4.8, all asphalt mixtures met the standard requirement of rut depth. Similar behaviour was observed in a previous study conducted by Ahmadinia et al. (2012) where shredded waste plastic was used in a dry process of SMA preparation. In a more recent study, Mahmoud, Shubber, and Jabur (2018) asserted that rut depth decreased with an increase in a waste aggregate replacement at 60 °C.

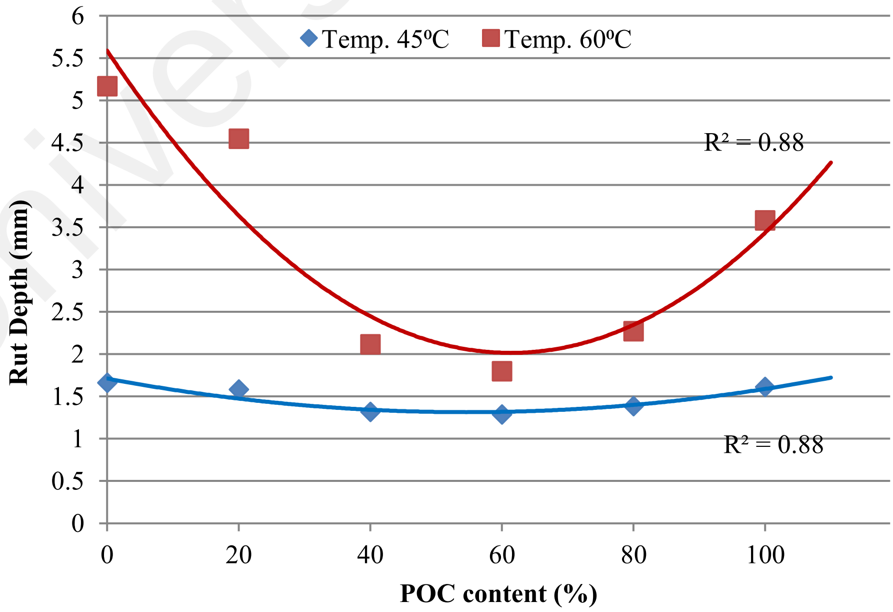


Figure 4.13: Rut depth of different mix proportions

Table 4.8: Normalised maximum rut depth of different POC mixtures

Mixtures	Temp. 45°C		Temp. 60°C	
	Rut Depth (mm)	Normalised Rut Depth (mm)	Rut Depth (mm)	Normalised Rut Depth (mm)
Control mix	1.66	1.00	5.17	1.00
POC-20	1.58	0.95	4.55	0.88
POC-40	1.32	0.79	2.12	0.41
POC-60	1.29	0.77	1.80	0.35
POC-80	1.39	0.83	2.27	0.44
POC-100	1.61	0.97	3.58	0.69

4.4.3 Dynamic Creep Test Results

Figure 4.14 illustrates the dynamic creep curves of different asphalt mixtures. A significant variance was observed between control and all SMA mixtures. All mixtures containing POC demonstrated lower permanent strain compared to the control mixture. This could be attributed to the presence of pozzolanic material in palm oil fuel ash which is composed of silicon dioxide (SiO_2) and aluminium oxide (Al_2O_3) (Karim et al., 2018; Nayaka et al., 2018). The pozzolanic material can increase the adherence of mixture compositions thus enhancing the mixture strength (Modarres & Rahmanzadeh, 2014).

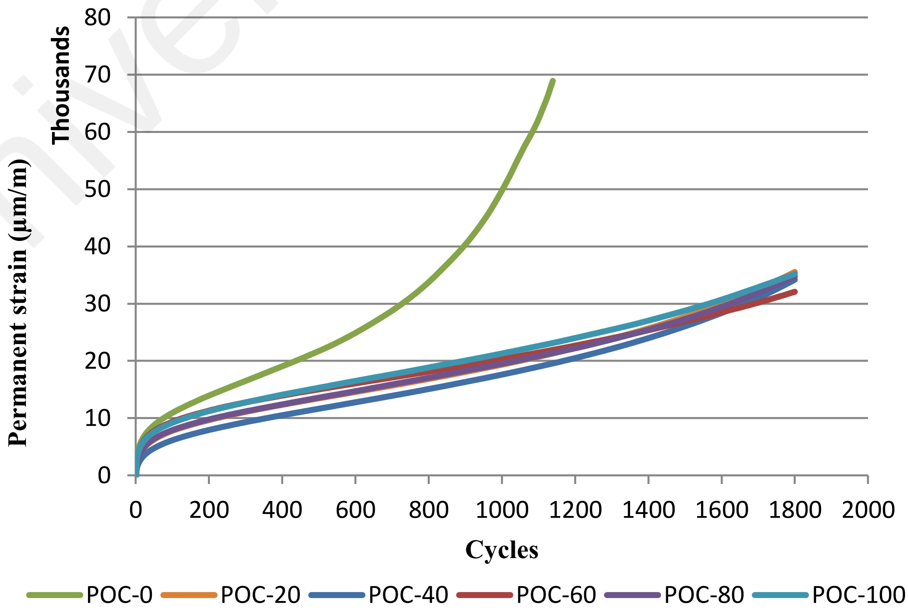


Figure 4.14: Permanent strain as affected by load cycles of different asphalt mixtures

The lower strain value recorded showed the mixture possessed higher rutting resistance. The cumulative strain and deformation of the final load cycle of asphalt mixtures are presented in Table 4-9. Asphalt mixture prepared with 60% of POC displayed a lower deformation of 1.62 mm and a strain of 32076 μm followed by the mixture containing 40% of POC which showed deformation of 1.73 mm and a strain of 34188 μm.

Table 4-9: Cumulative strain and deformation of different asphalt mixtures

Mixtures	Cumulative strain (μm/m)	Cumulative deformation (mm)
Control	68918	4.23
POC-20	35533	1.78
POC-40	34188	1.73
POC-60	32076	1.62
POC-80	34767	1.75
POC-100	35207	1.80

4.4.4 Indirect Tensile Strength Test Results

According to AASHTO-T283 AASHTO (2007), specimens should be compacted to 7.0% of air voids. To determine the suitable number of blows in the Marshall mix design, the relationship between air voids and the number of blows was evaluated and illustrated in Figure 4.15.

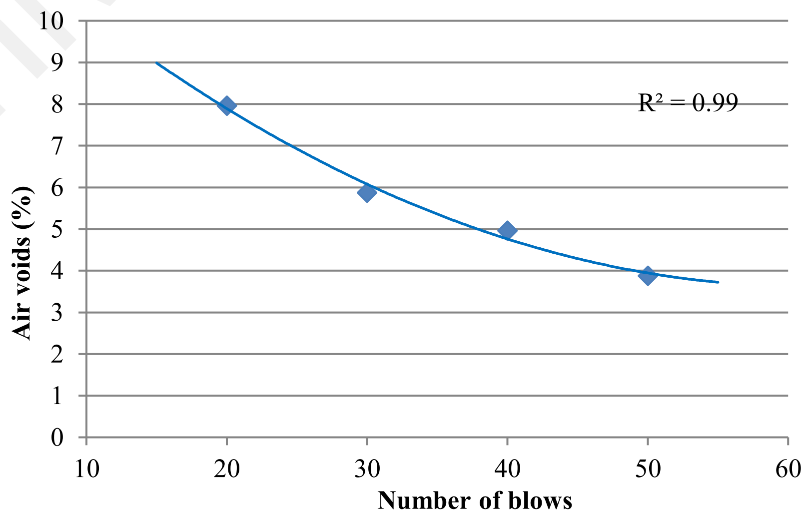


Figure 4.15: Percentage of air voids as affected by the number of blows

It was found that 25 blows can achieve 7.0% of air voids. The air voids of all compacted asphalt mixtures and control mix are presented in Table 4.10.

Table 4.10: Air voids in compacted asphalt mixtures and control mix

Sample	Dry set				Wet set			
	1	2	3	Average	1	2	3	Average
Control	6.64	7.17	6.67	6.83	7.02	6.58	7.12	6.91
POC-20	6.97	6.99	6.89	6.95	6.91	7.18	7.01	7.03
POC-40	7.04	6.71	6.73	6.83	7.24	6.54	6.65	6.81
POC-60	6.63	7.04	6.68	6.78	6.51	6.71	6.84	6.69
POC-80	6.77	6.75	6.93	6.82	6.60	7.42	7.00	7.01
POC-100	6.81	6.65	6.98	6.81	7.06	7.02	7.05	7.04

The average value of indirect tensile strength for different mixtures for both dry and wet conditions are portrayed in Figure 4.16 (See Tables A.20 to A.25 in Appendices for more details). Both conditions with POC replacement resulted in a better strength compared to the control mixture which could be due to the roughness of POC as fine aggregates. The POC aggregates are coarser than the natural fine aggregates. Coarse structure of POC increased the interlocking between the asphalt binder and the aggregates. Similar behaviour was reported by Ossa et al. (2016) where substitution with waste aggregate can decrease the stiffness of asphalt mixture for both wet and dry conditions.

Tensile strength ratio (TSR) was calculated by dividing the indirect tensile strength value for dry condition and the indirect tensile strength value for wet condition. The TSR of asphalt mixtures of dry and wet conditions are presented in Figure 4.17 . The value of TSR was found to increase with an increment in POC replacement until a maximum ratio was achieved at 93.1% of TSR which decreased subsequently.

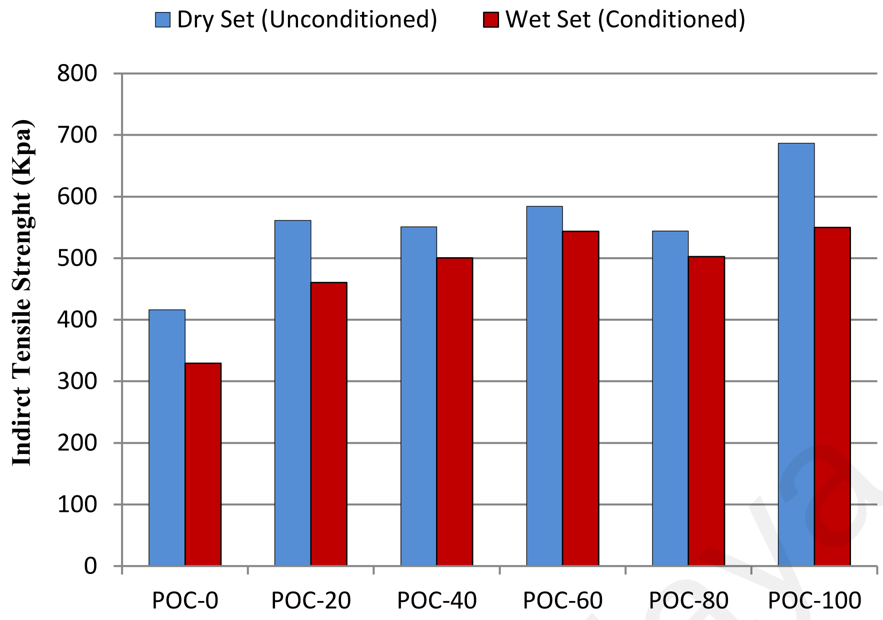


Figure 4.16: Indirect tensile strength of mixtures at dry and wet conditions

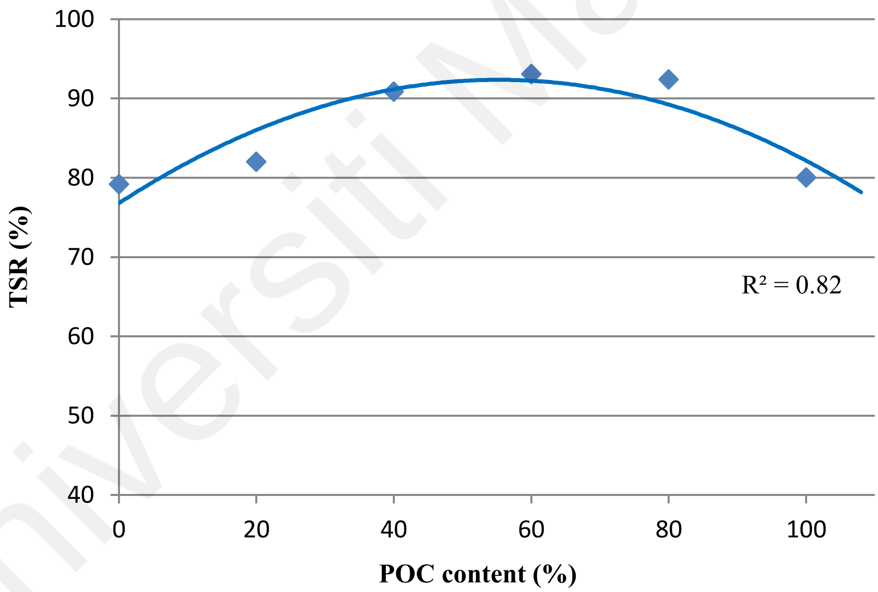


Figure 4.17: Tensile strength ratio of different mixtures

As shown in Table 4-11, all mixtures containing POC displayed greater TSR values than the control mixture. Mixtures that contained 40%, 60% and 80% of POC fine aggregates resulted in a higher TSR value with increment by 1.15, 1.18, and 1.17 times respectively compared to the control mix which indicated sufficient resistance against moisture damage. Besides, all the substituted aggregate mixtures were above the minimum required value (80%) as prescribed by AASHTO T 283 (AASHTO, 2007) and the

Malaysian Public Works Department for road works (Jabatan Kerja Raya (JKR) Malaysia, 2008).

Table 4-11: Normalised tensile strength ratio of different POC mixtures

Mixtures	TSR (%)	Normalised TSR
Control mix	79.2	1.00
POC-20	82.0	1.04
POC-40	90.9	1.15
POC-60	93.1	1.18
POC-80	92.4	1.17
POC-100	80.0	1.01

4.4.5 Cantabro Durability Test Results

Three samples for each group of mixtures were tested and weighed for every 100-cycle interval, and the results are tabulated in Tables A.26 to A.31 in Appendices. The average Cantabro loss for each interval was calculated and presented in Figure 4.18.

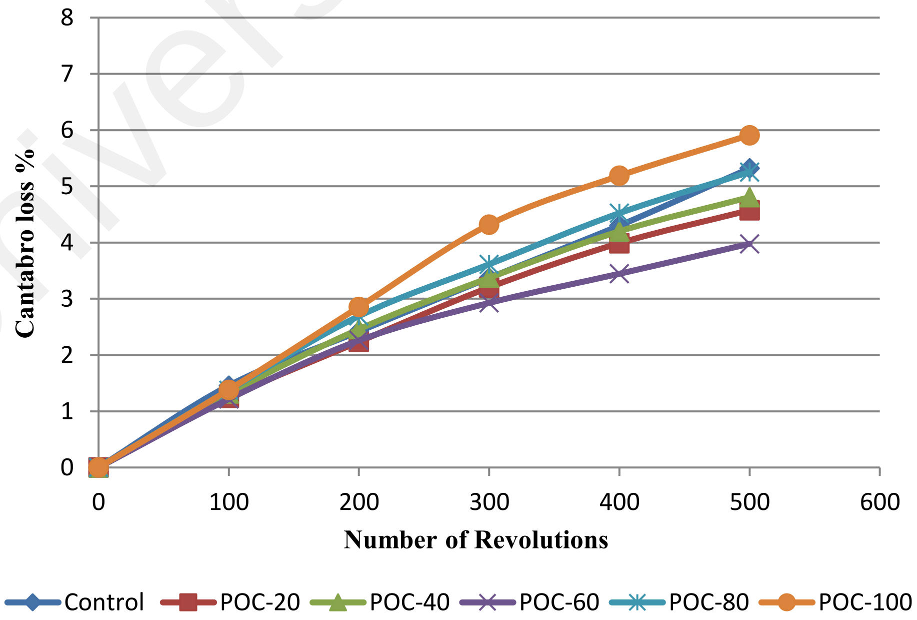


Figure 4.18: Cantabro loss as affected by the number of revolutions

Figure 4.18 illustrates a similar trend in Cantabro loss for all mixtures. As the number of revolutions increased, the mass loss of asphalt mixture also increased. The Cantabro loss values for different asphalt mixtures after 300 revolutions are portrayed in Figure 4.19. The Cantabro loss values decreased until a minimum value of 2.9% was achieved for POC-60 mixture, before a subsequent increase in the values.

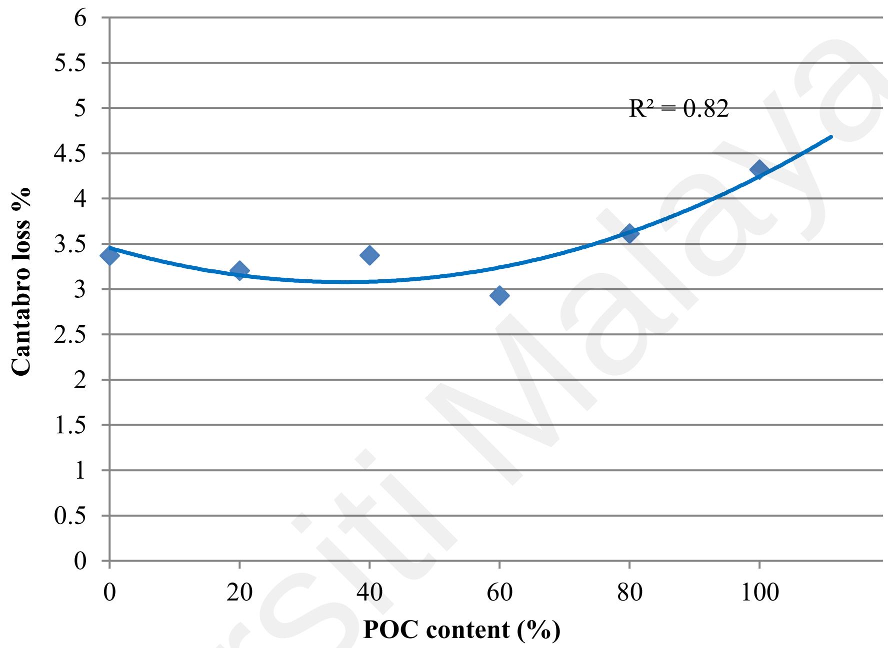


Figure 4.19: Cantabro loss for different asphalt mixtures at 300 revolutions

Based on the normalised values tabulated in Table 4-12, the asphalt mix replaced with 60% of POC was found to be the best mixture to resist abrasion, as the Cantabro loss decreased by 0.87 times as compared to that of control mixture. Based on the European Standard (EN 12697-17) (EN 12697-17, 2017) and JKR (Jabatan Kerja Raya (JKR) Malaysia, 2008) standard, the maximum values of weight loss for dry samples at 300 revolutions are 20% and 15% respectively. Table 4-12 shows that all mixtures satisfied the stipulated requirements. Meanwhile, the POC-80 and POC-100 mixtures showed a higher abrasion loss as compared to the control mixture. This could be related to porosity, surface texture, high content, and low strength of POC aggregates compared to the natural aggregates. So, high content of POC aggregates such as POC-80 and POC-100 mixtures contributed to increase the losses in weight.

Table 4-12: Normalised Cantabro loss at 300 revolutions

Mixtures	Cantabro loss (%)	Normalised Cantabro loss
Control mix	3.371	1.00
POC-20	3.202	0.95
POC-40	3.372	1.00
POC-60	2.926	0.87
POC-80	3.613	1.07
POC-100	4.318	1.28

4.4.6 Indirect Tensile Fatigue Test Results

During the fatigue test, sample deformation was recorded, and a curve was plotted corresponding to the number of load cycles. For each type of SMA mixture, three samples were tested, and the results were calculated and averaged.

The fatigue curves for all SMA mixtures with three different deformation zones are exhibited in Figure 4.20. For the first phase of displacement, the deformation increment rate was relatively high due to the compression of current air voids in the asphalt mixtures. The second phase is the elastic zone, where the low displacement increment can be observed with a linear fatigue curve. Moreover, the horizontal length of the second stage for SMA mixtures with POC replacement was significantly longer as compared to the control mixture. It was also found that the slope of the deformation curve in this stage was less than the control mixture. Therefore, the use of POC as a fine aggregate replacer enhanced the elastic property and exhibited higher cracking resistance than the control mixture, which possesses brittle fracture characteristics. Meanwhile, the third phase is called the plastic zone, categorised as the unstable phase where displacement increases rapidly due to crack growth in the SMA mixtures.

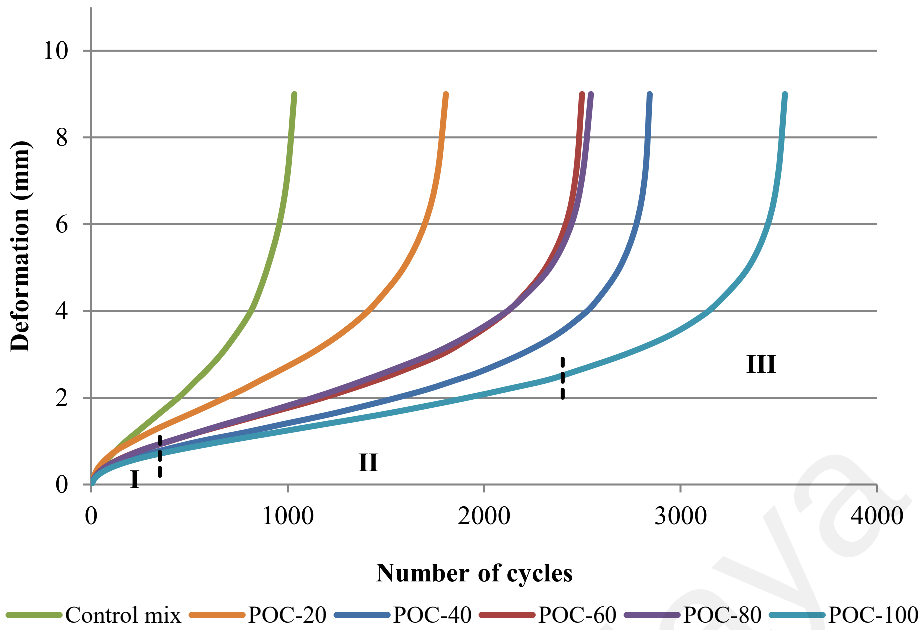


Figure 4.20: Deformation as affected by number of cycles for different asphalt mixtures

According to EN 12697-24 (British Standards Institution, 2018), fracture life is equal to the total number of cycles that led to the complete splitting of a sample. Figure 4.21 shows the fracture life for all the SMA mixtures with and without POC replacement. When compared with the control mixture, the fracture life of SMA mixtures with POC replacement increased. For example, the fracture life of SMA mixtures with POC replacement were 1.7 times (20% POC), 2.7 times (40% POC), 2.4 times (60% and 80% POC), and 3.4 times (100% POC) higher compared to the fracture life of control mixture (0% POC). This observation suggested that the fatigue performance of SMA mixtures can be enhanced by replacing the fine aggregates with POC. The use of POC improved the elastic properties, dispersion, and absorption of concentrated stress which were produced by fatigue loading. This delayed the progress of micro-cracks, which in turn postponed the asphalt sample failure. A similar trend of fracture life was observed in previous studies where SMA mixture was prepared using shredded waste plastic with a particle size of 2.36 mm (Moghaddam & Karim, 2012).

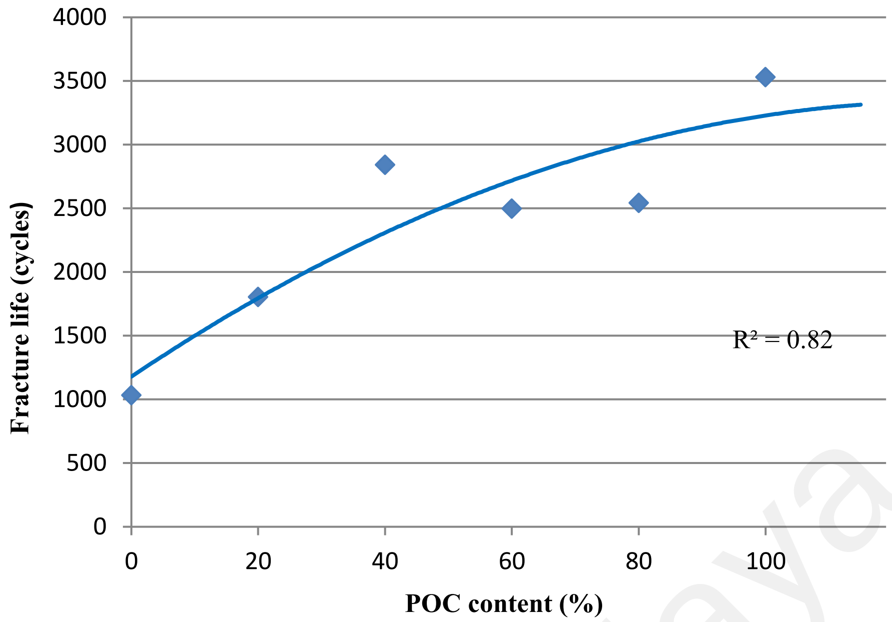


Figure 4.21: Fracture life of different asphalt mixtures

In the present study, regression analysis was carried out using SPSS to investigate the relationship between the asphalt mixture performance and POC content (%) and the results are summarised in Table 4-13. All the regression models obtained were of second-order polynomial and for most of the cases, the coefficients of determinations indicated strong relationships between the selected parameters.

Table 4-13: Regression models for asphalt mixture properties and POC content

Property	Regression models	R^2
Resilient modulus	Stiffness (MPa) = $-0.371 \times (POC)^2 + 43.52 \times (POC) + 7110$	0.932
Rutting (45°C)	Rut Depth (mm) = $1\times10^{-3} \times (POC)^2 - 0.116 \times (POC) + 5.59$	0.879
Rutting (60°C)	Rut Depth (mm) = $1\times10^{-4} \times (POC)^2 - 0.0146 \times (POC) + 1.712$	0.876
Dynamic creep	Strain (µm/m) = $3308.4 \times (POC)^2 - 28100 \times (POC) + 88289$	0.834
IDT (Dry)	Strength (kPa) = $-2.316 \times (POC)^2 + 54.3 \times (POC) + 402.2$	0.681
IDT (Wet)	Strength (kPa) = $-13.28 \times (POC)^2 + 129.24 \times (POC) + 230.2$	0.902
TSR	TSR (%) = $-5\times10^{-3} \times (POC)^2 + 0.563 \times (POC) + 76.8$	0.825
Cantabro durability	Cantabro loss (%) = $3\times10^{-4} \times (POC)^2 - 0.0208 \times (POC) + 3.46$	0.825
Fatigue	Fracture life (cycles) = $-0.129 \times (POC)^2 + 33.4 \times (POC) + 1178.9$	0.818

4.5 Overall Comparison and Ranking of Asphalt Mixture Performance

Different mix designs were ranked on a 6-point scale to determine the performance of mix design. The mixture with the best performance was ranked as 1 while the mixture with the least performance was ranked as 6 as shown in Table 4-14. Two different types of ranking which were (i) individual ranking, and (ii) overall ranking were used in this study. The individual ranking ranks asphalt mixture performance for selected performance test of asphalt mixtures conducted in this study which assisted in selecting the best mix design for each of asphalt mixture performance test. For example, for resilient modulus, asphalt mixture POC-40 was ranked 1 (the best performance), while the control mix was ranked 6 (the worst performance).

For overall ranking, the Relative Importance Index (RII) method (Akadiri, 2011) was used to calculate the relative significance of each mix design's rank for different performance tests. The RII was computed as follows:

$$RII = \sum \frac{1+A-W}{A*N} \qquad \text{……………………. (4.1)}$$

Where A is the highest weight = 6; W is the weight given for each performance test which ranged from 1 to 6 based on the performance rank; and N is the total number of performance test.

The RII was calculated and presented in Table 4-14. Based on the RII values, the overall ranking of asphalt mixture performance was determined. Asphalt mixture (POC-60) was ranked as the mixture that exhibited the best performance (RII = 0.90) followed by asphalt mixture POC-40 (RII = 0.77).

Table 4-14: Asphalt mixture performance ranking

Performance test	Mix design					
	Control	POC-20	POC-40	POC-60	POC-80	POC-100
Resilient Modulus	6	5	1	2	3	4
Wheel Tracking@45	6	4	2	1	3	5
Wheel Tracking@60	6	5	2	1	3	4
Dynamic Creep	6	4	2	1	3	5
Indirect Tensile Strength	6	3	4	2	5	1
Tensile Strength Ratio	6	4	3	1	2	5
Cantabro Durability	4	2	3	1	5	6
Indirect Tensile Fatigue	6	5	2	4	3	1
Relative index	**0.21**	**0.50**	**0.77**	**0.90**	**0.60**	**0.52**
Overall Ranking	**6**	**4**	**2**	**1**	**3**	**5**

4.6 Relationship between Performance of Asphalt Mixtures

To investigate the relationship between the performance of the asphalt mixture, Pearson correlation test was carried out. Specifically, the linear relationship between (i) rutting and (ii) fatigue with resilient modulus, indirect tensile strength, TSR, dynamic creep and Cantabro durability performance were determined. The correlation coefficient, R, and P-value of all asphalt performance tests are summarised in Table 4.15.

Table 4.15: Correlation matrix of different asphalt mixture performances

			Resilient Modulus				Indirect Tensile Strength		TSR	Dynamic Creep	Cantabro Durability
			5˚C	25˚C	35˚C	40˚C	Dry	Wet			
Rutting	45˚C	R	0.399	-.938*	-0.111	-0.031	-0.154	-0.571	-0.975*	0.616	0.449
		P- value	0.434	0.006	0.834	0.954	0.771	0.236	0.001	0.193	0.372
	60˚C	R	0.100	-.963*	-0.420	-0.348	-0.395	-0.754	-0.926*	0.718	0.154
		P- value	0.851	0.002	0.406	0.499	0.438	0.083	0.008	0.108	0.771
Fatigue		R	0.465	0.623	0.795	0.824*	.894*	.900*	0.309	-0.759	-0.759
		P- value	0.352	0.187	0.059	0.044	0.016	0.014	0.551	0.080	0.080

* Correlation is significant at the 0.05 significance level.

Correlation coefficients and P-values shown in Table 4.15 indicated that some mechanical performances of asphalt mixtures demonstrated good linear relationship with each other. Significant correlations between each pair of asphalt mixture performance are explained in the following subsections.

4.6.1 Rutting

A strong correlation was noted between rutting (measured at 45 ºC and 60 ºC), and resilient modulus (measured at 25 ºC) and tensile strength ratio of SMA mixtures with POC replacement as shown in Figure 4.22 and Figure 4.23, respectively. From Table 4.15, negative values of Pearson correlation coefficients (R) between rutting and resilient modulus were observed which indicated when the rutting values of SMA with POC replacement increased, the resilient modulus and tensile strength ratio decreased.

As can be seen from Figure 4.22 and Figure 4.23, coefficients of determination were high ($R^2 > 0.85$) which indicated rutting performance aligned/agreed with the mixture stiffness performance. According to a previous study conducted by Mashaan (2016a), there was an acceptable correlation ($R^2 = 0.71$) between rutting and resilient modulus of asphalt mixture.

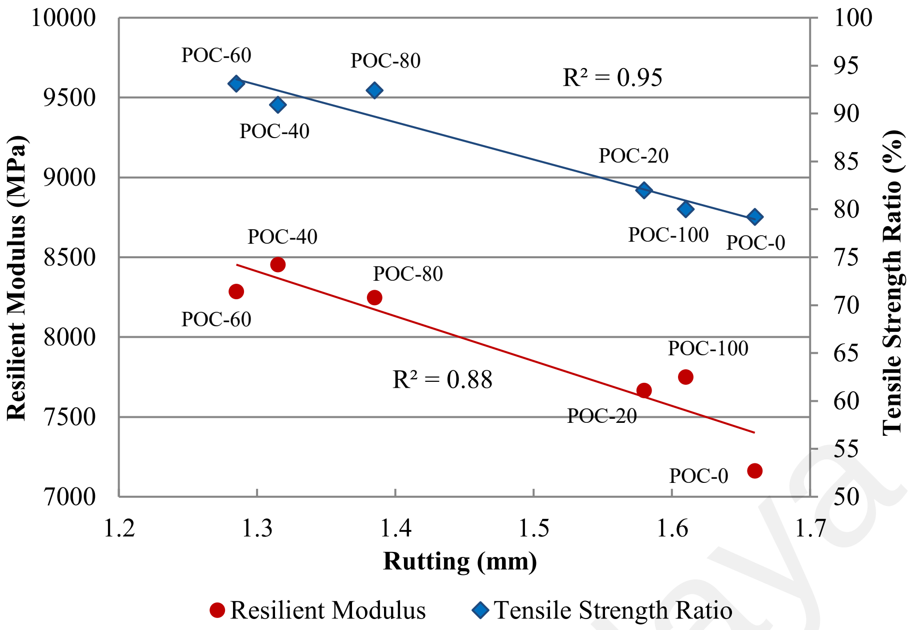


Figure 4.22: Relationship between resilient modulus, TSR, and rutting at 45 ºC

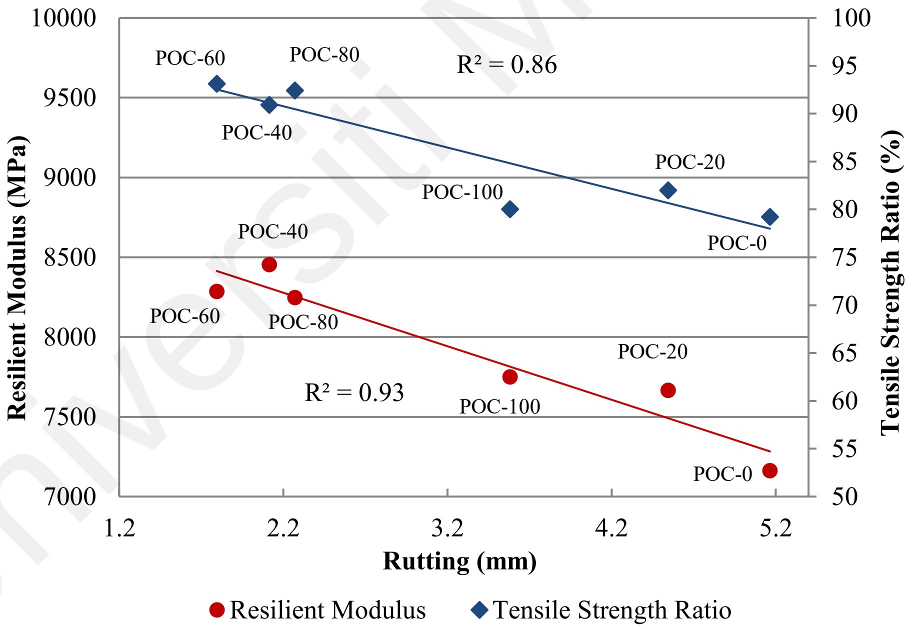


Figure 4.23: Relationship between resilient modulus, TSR, and rutting at 60 ºC

4.6.2 Fatigue

Figure 4.24 exhibits the correlation between fatigue and indirect tensile strength, while Figure 4.25 displays the correlation between fatigue and resilient modulus measured at 40 ºC.

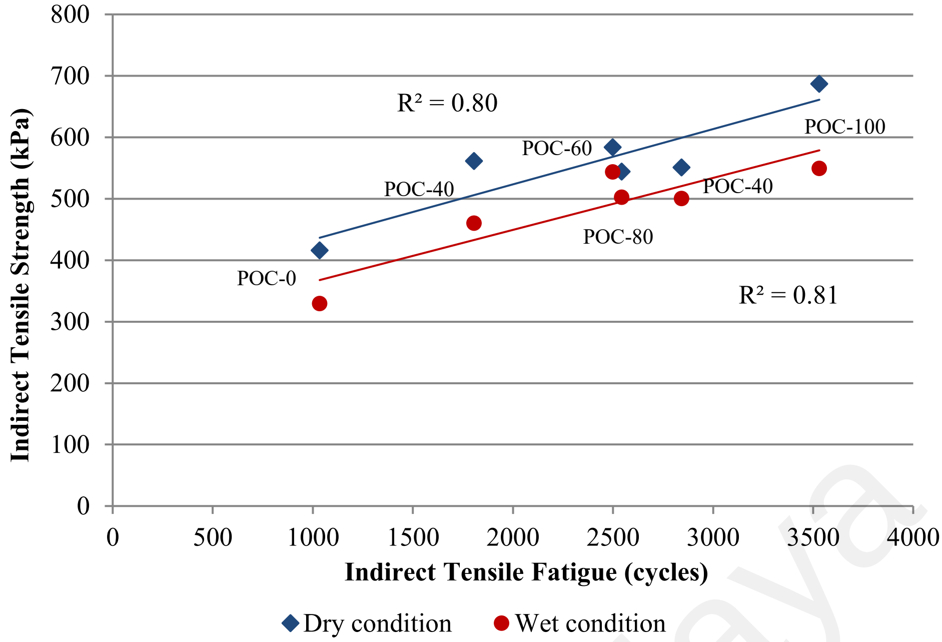


Figure 4.24: Relationship between indirect tensile strength (ITS) and fatigue

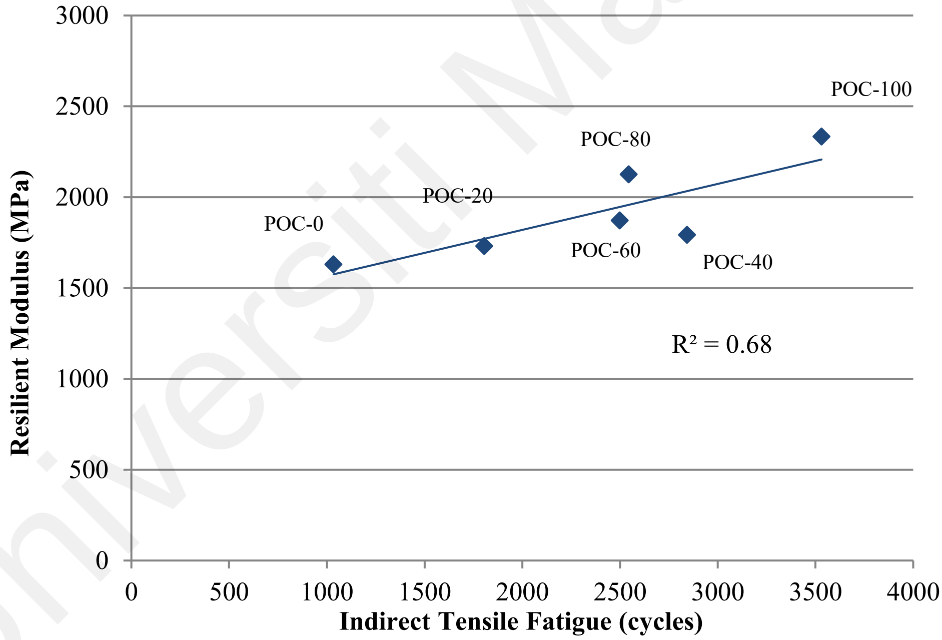


Figure 4.25: Relationship between resilient modulus and fatigue

Pearson correlation coefficients for the relationship between fatigue and indirect tensile strength for dry and wet conditions were 0.89 and 0.90, respectively, while the Pearson correlation coefficient was 0.82 for the relationship between fatigue and resilient modulus (Table 4.15). The results indicated positive correlation, where an increase in fatigue performance of SMA with POC replacement caused an increase in indirect tensile strength performance, and resilient modulus of both conditions as shown in Figure 4.24 and Figure 4.25, respectively.

Hamed (2010) found a good correlation between fatigue life of asphalt mixtures and resilient modulus when crumb rubber and SBS modifiers were used. Similar behaviour was found by Mahrez, Karim, and bt Katman (2005) where asphalt mixture with reinforced by glass fibre was researched. Moreover, A strong linear relationship (R^2 = 97) between the resilient modulus and fatigue life was found in the previous study by Modarres et al. (2015).

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

The main aim of this research was to study the effect of POC contents on the mechanical properties of SMA mixture. Therefore, different types of laboratory experiments were carried out to evaluate the mechanical properties of SMA mixture containing different levels of POC. Based on the experimental results and statistical analyses performed, the replacement of fine granite aggregate with POC influenced the mechanical properties of asphalt mixtures. The following conclusions are deduced followed by recommendations for future research:

5.1 Conclusions

1. From the Marshall test results, it can be concluded that POC is suitable to be used as a fine aggregate replacer up to 100% in the SMA mixture and can satisfy the mix design requirements.

2. Optimum binder content for all the different mixtures was 6.14 ± 0.1% of the total weight of the mixture. No statistical significant difference was detected for optimum asphalt contents of all mixtures.

3. The resilient modulus values increased with increasing POC content until reach to the maximum (at 40% of POC replacement) and then decrease. Moreover, all the SMA mixtures with POC replacement showed higher resilient modulus values compared to the control mix.

4. For wheel tracking performance, the permanent deformation values for all SMA20 mixtures with POC replacement showed lower values compared to the control mix at 40 °C and 60 °C. This indicated that the replaced mixtures with POC showed higher rutting resistance as compared to the control mixture.

5. For dynamic creep performance, the permanent strain curves showed a distinct trend between the control mixture and SMA20 mixtures with POC replacement. SMA20 mixtures with POC replacement showed a lower permanent strain and rut depth as compared to the control mixture.

6. For indirect tensile strength performance, all SMA20 mixtures with POC replacement indicated better strength as compared to the control mixture for both dry and wet conditions. Moreover, the SMA20 mixtures have higher TSR values as compared to the control mixture. This finding indicated that mixtures with POC replacement have sufficient resistance to moisture damage. Furthermore, all the mixtures with POC replacement demonstrated TSR higher than the minimum required value, i.e. 80% of TSR.

7. For Cantabro durability performance, all the mixtures showed a similar trend. As the number of revolutions increased the mass loss decreased. With the exception of the mixtures with 80% and 100% POC replacements, the SMA mixture with POC replacement showed slightly lower Cantabro loss as compared to the control mixture. Conclusively, all the mixtures fulfilled the requirement of the Cantabro durability maximum value (i.e. 20% and 15%) of the weight loss as outlined in the European and JKR standards.

8. As indicated by indirect tensile fatigue test finding, the horizontal length of the second stage of SMA mixtures with POC replacement was significantly longer, and the slope of deformation curve in this stage was less than the control mixture. Hence, adding POC to SMA mixtures enhanced the elastic properties and exhibited higher cracking resistance. The fracture life of asphalt mixtures also increased with increment of POC replacement.

9. Based on comparison and ranking analyses of asphalt mixture performance, the optimum POC content in SMA mixture was proposed to be 60%. SMA mixture with this content showed the best mechanical properties performance as compared to SMA mixtures with other POC levels.

10. Based on the findings, the use of POC in highway pavement construction was proved to be one of the feasible methods in solving pollution issue related to POC disposal. The POC is suitable to be applied as a fine aggregate replacer until 100% in the SMA20 mixtures. These findings can contribute to the reduction in the negative impact of wastes on the environment and lead to green, sustainable, and environmentally friendly pavement design.

5.2 Recommendations for future research

The recommendations to improve this research are listed as follows for researchers interested in asphalt mix design and flexible pavement materials area:

1. In the present study, fatigue performance of asphalt mixture was investigated using indirect tensile fatigue test method. This test was used due to the unavailability of equipment for four-point bending fatigue test. For future research, fatigue property is recommended to be investigated using a four-point bending fatigue test (beam fatigue test).

2. This study evaluated the mechanical performance through resilient modulus, wheel tracking, dynamic creep, indirect tensile strength, Cantabro durability, and indirect tensile fatigue. For future research, a long-term durability test is recommended to be investigated.

3. A field trial test for the proposed mixes is recommended to validate the laboratory results reported in this study. This is significant to ensure the results obtained from laboratory works are consistent with the results from the field testing.

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